YOURS

PROCEEDINGS

Vth WORLD CONGRESS ON WATER RESOURCES BRUSSELS 1985

VOLUME 2

WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

PROCEEDINGS

VOLUME 2

of the

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9-15 JUNE 1985 Brussels Belgium

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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**.

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 Kosinsky

L'Organisateur Scientifique

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9 - 15 JUNE 1985 BRUSSELS BELGIUM
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Paper number 137 Aspect number 10

EAST MPANDA RURAL DEVELOPMENT PROJECT

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ABSTRACT

The East Mpanda region is the easterly part of the Imbo plain, north of Bujumbura, Burundi. It covers some 12,600 ha.
The farmers there cultivate mainly cotton, sorghum and maize on rainfed plots, and rice on irrigated fields. The present irrigation system, covering a total of 1500 ha, dates back to the 1950's and is in urgent need of rehabilitation. This fact was confirmed by a survey carried out by the Consultant in 1981, which showed that only about 750 of these hectares are now actually irrigated. The reason is either inadequate maintenance of the irrigation system or frequent flooding of the fields by rivers that top their banks in the rainy season. The Government intends to rehabilitate and to extend the areas under irrigated cultivation to 2100 ha in order to increase the agricultural output.
A feasibility study for a development project was carried out in 1976-1977. This was followed by further studies and preparation of detailed design and tender documents in 1981-1982. The execution of the works started in 1984. The project will be operational in 1987. Financing of the project is guaranteed by loans from AfDB, IFAD, OPEC, EDF, WFP and the Government of Burundi.
The studies revealed that two crops a year could be grown if night storage reservoirs were used. The proposed crops are rice (three varieties), vegetables and coffee. The proposed irrigation system consists of a gravity system fed by rivers via dams and intake works. Canals need to be lined, as a smaller cross-section will reduce losses and construction costs.
The overflowing of river banks is to be checked by constructing dikes along the rivers.
It has been recommended to upgrade the road system that was constructed in the plain in the 1950's and to construct new roads to serve more farmer families.
Social economic studies have been carried out on the unregulated immigration from other regions of Burundi. The influx of people has led to a need for new social-infrastructural works, such as a drinking water supply throughout the Imbo plain and village centers comprising schools, dispensaries, a hospital, sheds for storing produce.

EAST MPANDA RURAL DEVELOPMENT PROJECT

INTRODUCTION

In an effort to boost food production and to improve the welfare of the people, the Government of Burundi has embarked upon several development projects in the Imbo plain. One of these is the East Mpanda Rural Development Project, which is the subject of this article.

First a brief description is given of the country, the project area and the population, after which the technical aspects of the project are considered in more detail.

The Government has charged the Société Régionale de Développement de l'Imbo (SRDI) with the execution of the project.

THE COUNTRY

Burundi, one of the smallest countries in Africa, is located just south of the equator. The country has common borders with Rwanda to the north, Tanzania to the east and south and Zaire to the west. It covers an area of some 28,000 km^2 and has 4.1 million inhabitants (1979). The border with Zaire is formed by the Ruzizi River and by Lake Tanganyika, one of the largest lakes in Africa.

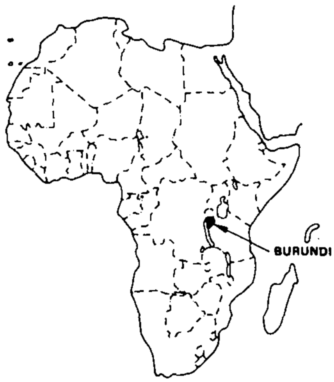
Geographically, the country may be divided into four zones:

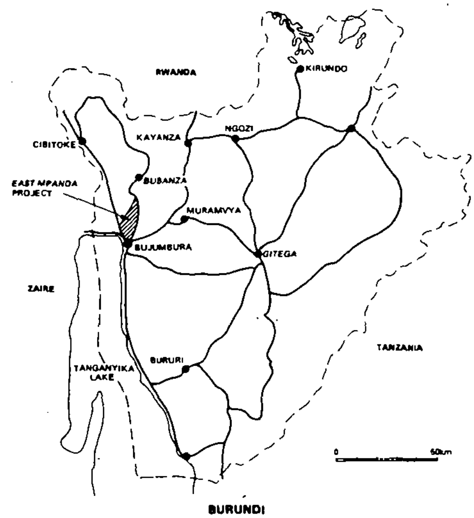
- i The Imbo plain in the west which comprises the Ruzizi valley and the shores of Lake Tanganyika. This area has a relatively hot and humid climate.
- ii The Zaire-Nile crest, east of the Imbo plain, where Burundi's highest peaks (up to 2670 m) can be found. This region has a considerably cooler climate than the rest of the country.
- iii The moderately high relief area to the east of the crest; this comprises the Ngozi, Kayanza, Gitega and Muramvya regions, with altitudes between 1500 m and 2000 m, where the climate is moderate.
- iv The so-called central plateau, lying further to the east and south-east, with a similar climate to that of the Imbo plain. One of the plains in this area is called the Mosso plain.

The capital Bujumbura, located at the north eastern tip of the lake, has some 160,000 inhabitants.
The second town of the country is Gitega, with 10,000 inhabitants. These figures make it clear that the large majority of the people live in rural areas. The average population density is 150 people per km^2, but this figure may rise to 300 in some regions. Burundi is the most densely populated country of Africa and possibly of the world. The population growth rate is currently 2.2 per cent per annum.

The Imbo plain, with which this article is mainly concerned, is relatively sparsely populated despite the region's agricultural potential. The majority of the country's inhabitants live east of the Zaire-Nile crest where climate and living conditions are considered better.

Burundi is an agricultural country characterized by a high population growth and critically limited arable lands. The people live mainly in the rural areas although the level of the facilities there is rather low. Education and health services, water supply and electricity are inadequate and there is insufficient employment in the non-agricultural sectors.





Some years ago the Government, aware of the seriousness of this situation, decided to improve the welfare of the population by implementing a number of social and infrastructural projects. The plans were aimed at stepping up food production, developing the manufacturing industry to create more jobs and opening up the country by road construction.

Two activities were specifically undertaken by the Government to tackle the problem of undernourishment and overpopulation in the hilly areas. These were:

a. intensification of cropping in the overpopulated regions through training and extension services
b. development of the less populated regions such as the Imbo and Mosso plains

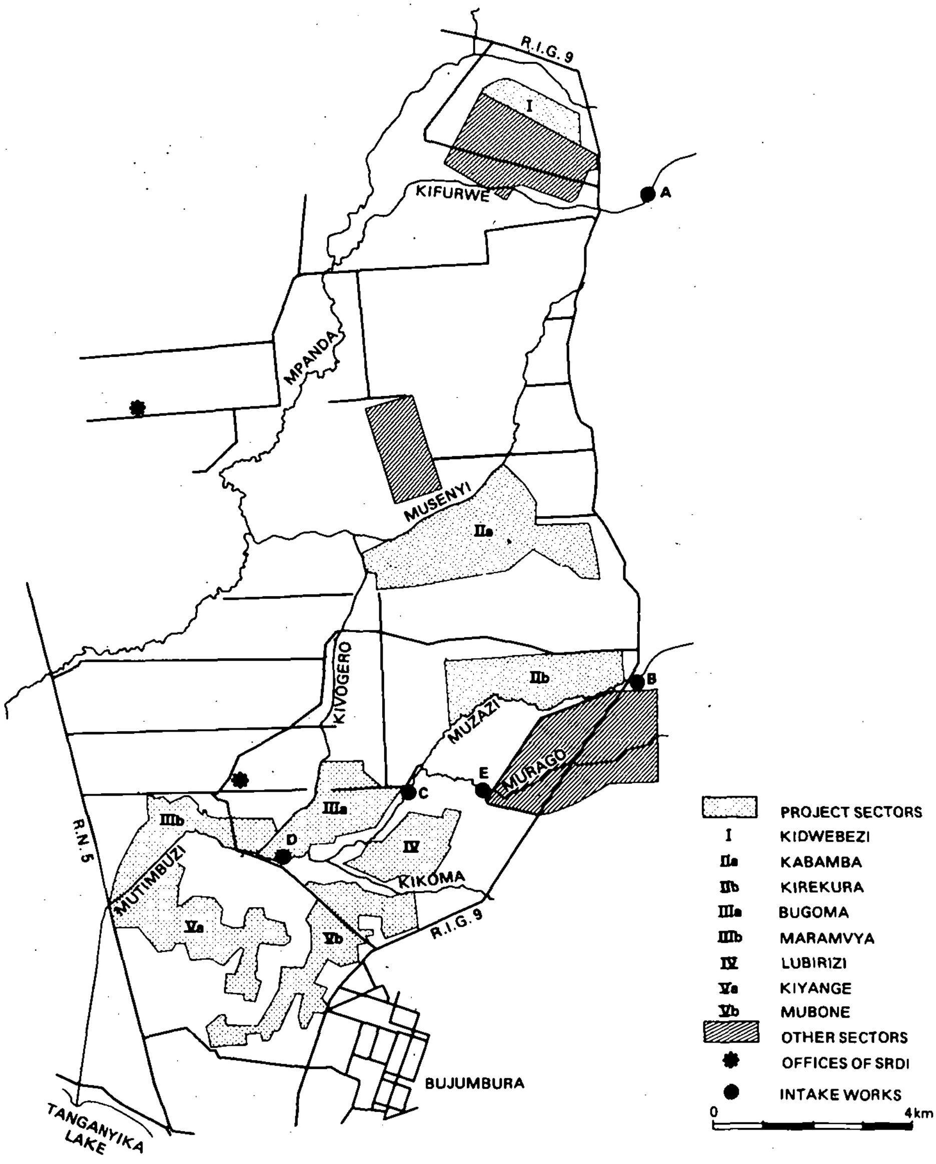
Several projects of this kind have already been carried out.

It should be mentioned that a major part of Government funds come from the export of coffee. The drop in coffee prices on the international market had a serious effect on the country's economy in 1978 and 1979. It is therefore important that the country diversifies its sources of export earnings and starts growing more of its own food crops.

These objectives are clearly valid for the East Mpanda Rural Development Project. Firstly, the project aims at increasing food production and secondly at stepping up the standard of living of the people in this region.

THE PROJECT AREA

The project area is located in the south-eastern part of the Central Imbo plain, just north of the capital.
The area is bordered by the Mpanda River and by national road No. 5 (to the west), by the foothills of the Zaire-Nile crest or more precisely by regional road No. 9 (to the east), and by Lake Tangayika and the town of Bujumbura (to the south).



EAST MPANDA PROJECT AREA

Its location east of the Mpanda River has given it its name. The total area covers some 12,600 ha.

The climate in the area is characterized by a rainy season that lasts from October to May, with a dry spell in January and February. Thus the period from June to September is dry. The heaviest rainfall is in March and April. The average annual rainfall is 850 mm and the potential evaporation has been estimated at 1700 mm. The temperature remains fairly constant, at about 24°C.

The rivers traversing the project area on their way to Lake Tangayika are (from north to south) Kifurwe, Musenji, Muzazi/Mutimbuzi, Murago and Kikoma. Occasionally parts of the area are flooded by high waters from one of these rivers. All irrigation water required for the project will have to be taken from these rivers. The water from the Mpanda River has been reserved for other areas and projects.

The soils in the project area are mostly fertile. Classified by the USBR standard, half the area is class 2, a quarter is class 3 and a quarter class 4.

The area may be divided into three zones in regard to kinds of agricultural activity:

a. Peasant cotton fields where each farmer has 4 ha on which to raise cotton, maize and crops for personal use; the farmer lives on his plot. The plots were set out along roads built in the form of a grid, at distances of 1.2 km ("transversales").
b. The irrigated rice fields of Bujumbura, constructed in the 1950's, covering a total area of some 1500 ha; the irrigation water was to be drawn from some of the rivers mentioned above; due to neglected maintenance the fields and the infrastructure have deteriorated.
c. The areas outside the peasant fields; here some rainfed agriculture has been practised.

POPULATION

When the project was initiated in 1976 it was assumed that the number of families in the area should be raised so that the whole area could be cultivated.
It was believed that 3475 families should move from the (overpopulated) hills to the plains to arrive at the number of 5320 families, considered necessary to cultivate the area according to the first plans.

In 1982 and 1983 a detailed socio-economic survey was carried out to forecast the effects of the project. As part of the survey, a population census was conducted. This census revealed that some 6500 families were currently installed in the area, 5500 of them being engaged in agricultural activities. Clearly, the planned influx of people was no longer necessary.

The population growth has been estimated at an average of 10 per cent over the past three years, although a spatial variation can be discerned. In the south of the area the growth reached values of up to 20 per cent per year whereas in the north this was only 4 per cent. As already stated, the country-wide average is 2.2 per cent.

At the commencement of the project it was feared that the insalubrity of the plains would keep people away and discourage settlement. However, this has proved to be far from the case. Two major reasons that may explain this fact are the overpopulation of the interior of the country and the attraction of living near the capital.

In conformity with the population growth figures, it was found that the size of the holdings is rather small in the south (e.g. Lubirizi 1 ha) and large in the north (e.g. Kidwebezi 8 ha). The average size is 2 ha.
The sociological survey revealed that the population density increases from north to south and from west to east.

Although people may no longer need to be moved from elsewhere to the plain, to increase the numbers there, it may be necessary to encourage them to move within the project area. This is because the Government wants to build a number of new villages around the irrigation projects that are to be created.

THE PROJECT

The main purpose of the East Mpanda Rural Development Project is to increase agricultural production and to improve the welfare of the people in the region. A number of activities and projects will have to be implemented in order to achieve these aims.

Only the purely technical aspects are considered in this article. Other aspects, such as the health improvement programme and the food assistance programme, lie beyond its scope.

The main technical elements of the project are
- the flood protection works
- the irrigation system
- the road network
- the water supply system
- the electricity supply
- the construction of offices, etc.
- the construction of hospitals and schools

The technical details of each of these are briefly described in the following paragraphs.

Flood protection works

A number of rivers flow through the East Mpanda region. The most important of those are the Muzazi or Mutimbuzi, the Musenyi and the Murago, all of which are perennial rivers.
Studies carried out by the Consultant indicated that the discharges of the rivers would be sufficient for the irrigation planned in the project area. However, due to heavy erosion in the hills that form their catchtments, the rivers carry large amounts of sediments. As they enter the Imbo plain the slope of the riverbed changes abruptly to become rather flat. Consequently the velocity of the water diminishes and the sediments settle on the riverbed. This process has worsened in recent years, so that the riverbed has become steadily higher. As a result the river has breached its banks, flooding large areas of agricultural land.
In the course of the project it has been proposed to construct dikes along the rivers, to overcome the problem of flooding.
An alternative plan, to divert the flood water to another gully in the plain, proved unfeasible since the gully slope was not sufficient to carry all the sediment to Lake Tanganyika.
Dikes will be built along the Musenyi (left bank only), Muzazi/Mutimbuzi (both banks between RIG 9 and RN 5) and Murago (both banks, from 500 m upstream of the confluence with the Muzazi).

Agriculture and irrigation

In the 1950's the farmers in the Imbo plain were encouraged to change their farming methods by adopting a new arrangement of their fields, the so-called "paysannats". Each farmer was allotted a "paysannat", measuring 4 ha and divided into 10 parcels. Two of these parcels were used for the construction of a house and to grow subsistence crops, the remaining eight were to be cultivated a in pre-set cropping pattern with cotton, maize and sorghum, three parcels being left fallow each year.
In another part of the Imbo plain, on an area of 1500 ha near Bujumbura, an irrigation system was constructed in the 1950's. Here each family was allotted a plot of 1.5 ha including three parcels destined for rice, vegetables and lying fallow. The farmers did not live on their fields.
However, at the time of the surveys for the project design, it was found that only about half the area was under cultivation. Malfunctioning of the irrigation system must be considered the main circumstance leading to this situation. In the first place, the major intake structures in the system were destroyed by floods in 1962 and 1964. The farmers then improvised the intake of water by means of simple structures made of branches, stones etc. These structures were washed away with each major flood, so irrigation was interrupted and the improvised intakes constantly had to be rebuilt.
As an element of the East Mpanda project it is planned to upgrade part of the Bujumbura rice fields, covering approximately 1200 ha. In addition three other areas, totalling some 900 ha, are to be newly developed (see Table 1).

Table 1 - Irrigation sectors

Sector	Surface ha	Intake	River
Bujumbura rice fields			
Bugoma	185	C	Mutimbuzi
Maramvya	195	C	Mutimbuzi
Kiyange	335	D	Mutimbuzi
Lubirizi	185	E	Murago
Mubone	315	E	Murago
New areas			
Kidwebezi	95	A	Kifurwe
Kabamba	510	B	Muzazi
Kirekura*	275	B	Muzazi
Total	2095		

* In view of the results of the socio economic studies, it was decided not to construct the irrigation system for the Kirekura sector where coffee would have been grown.

Water is to be supplied from the rivers to the fields by means of a gravity supply system. It will be diverted by a low level dam, passing a sand trap before entering the main canal. The design discharges of the five off-take structures are given in Table 2.

Table 2 - Design discharges

Intake see fig.	Max. discharge m^3/sec.	River
A	0.09	Kifurwe
B	0.51	Muzazi
C	0.37	Mutimbuzi
D	0.49	Mutimbuzi
E	0.28	Murago

The main canals are in general to be lined to reduce losses. However, the necessity of lining will depend on local soil conditions. At some places the soils are rather sandy, making lining essential.

The amount of water in the rivers is not sufficient to grow two crops a year. However, if a night reservoir is built at the head of the system, the night flow can be stored for use during the day. Night operation of the system is unfamiliar to the local farmers though.

Beans and rice will be grown in soils of class 2 and 3; on soils of class 4 there are two alternatives: alternative I two rice crops and alternative II one rice crop and a vegetable crop on one third of the area.
The irrigation system will be so designed that both alternatives are possible.

Roads

The road system inside the project area was also constructed in the 1950's. It consisted primarily of a system of roads running east-west at distances of 1260 m. These roads were called transversals. One or two other roads were constructed from north to south to interconnect the transversals.
The two roads around the area, national road no. 5 to the west and regional road no. 9, do not form part of the internal road system. These roads are maintained by the Ministry of Public Works.

Some roads inside the area have been maintained to a reasonable standard, since they serve important population areas. Other roads however, have not been maintained and are consequently in urgent need of repair and upgrading, particularly since one of the objectives is to raise the standard of living of the people.
The Consultant has proposed to (re)construct the roads so as to have a width of 5 m. The roads would be given a top layer of laterite or sand depending on the subsoil. The length of the road network is 126 km.

Domestic water supply

The region has a water supply system that is supplied by a number of natural springs in the hills near Muzinda, near the Kirekura sector.
The yield of these springs was not sufficient to supply the growing number of people. It was therefore proposed, in the course of the evaluation of the project, to sink a number of shallow wells in the plain in order to supply the increased population and to serve other areas within the region.
During his surveys of the region the Consultant discovered a number of new springs with a sufficient yield. It has been proposed to include these new springs in the existing scheme, which can then be expanded.
The new scheme has been designed to supply the people with 45 litres per head per day.

At regular intervals (approximately 500 m) along the roads in the area small reservoirs (of 3 m^3 or 6 m^3) have been or will be constructed, each provided with one or two taps. These reservoirs are connected to the springs by a network of pipes. The water is supplied untreated.

Connections to houses have not been planned, but the maximum walking distance to a tap is 250 m. In large communities larger numbers of reservoirs will be constructed. As a rule of thumb it has been planned to construct a 3 m^3 reservoir per 50 families.

Electricity supply

The project includes an electrification programme for the offices of the SRDI. A high tension line is to be constructed between Bujumbura - Maramvya and Mugerero. Stand-by generators are also to be installed in accordance with the evaluation report of the IFAD.
The electrification programme does not include electricity supply to the villages in the project area, because of the high cost involved.

Construction of buildings

At present the offices of the SRDI are located at Mugerero, west of the Mpanda River. Stores for produce and a workshop for the SRDI vehicles are also located there. Since this compound is rather far away from the East Mpanda region it was decided to construct a new office at Maramvya, from which the project operations would be directed.
At the same time the office at Mugerero will be enlarged and a vehicle workshop constructed near the rice factory in Bujumbura.

The project will also contribute to the social infrastructure by constructing a school, a store and a dispensary in a number of villages. A hospital will also be built in the area near one of the new irrigation schemes.
At present the region has a limited number of these facilities. The socio-economic survey revealed that additional facilities are urgently required. The Table 3 shows the number of existing facilities and those planned.

Table 3 - Number of facilities

Facilities	Required	Existing	To be constructed
School	10	5	5
Store	7	2	5
Dispensary	4	2	2
Hospital	1	0	1

FINANCING THE PROJECT

In recent years the project has been through the following stages a) a pre-feasibility study (1976-1977); b) an appraisal mission by the African Development Fund and the International Fund for Agricultural Development (1979); c) preparation of detailed designs and tender documents (1981-1982); d) socio-economic and health survey (1982-1983); e) construction of the works (commenced in 1984, expected to continue until 1987).

Financing of the project has been guaranteed by the African Development Fund (AfDF), the International Fund for Agricultural Development (IFAD), the Organization of Petroleum Exporting Countries (OPEC), the World Food Programme (WFD), the European Development Fund (EDF) and the Government of Burundi.

The total project costs are estimated at approximately 30 million US dollars. A break down of the project costs is given in Table 4.

Table 4 - Estimated project costs

	millions of US dollars (current value)		
Foreign aid			27
- civil works		19.5	
river training works	5.9		
irrigation works	3.8		
domestic water supply	1.0		
electricity supply	0.7		
road construction	3.8		
offices etc.	1.6		
social infrastructure	2.7		
- consultant services		2.8	
- health programme		2.6	
- food aid		0.3	
- supply of equipment		1.8	
local contribution			3.5
- SRDI (personnel and running cost)		3.0	
- houses in new villages		0.2	
- farmers' contribution		0.3	
Total			30.5

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 138 Aspect number 4

LARGE-SCALE IMPLEMENTATION OF WATER SUPPLY SCHEMES FOR SUB-DISTRICT CAPITALS (IKK) IN INDONESIA; A STANDARDIZED APPROACH

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ABSTRACT

In 1981 the Indonesian Government initiated a nation-wide campaign, aimed at supplying a major part of the Indonesian population with drinking water before 1985.

To achieve this objective, the so-called IKK programme was initiated, covering the water supply to approx. 3000 sub-district capitals in the coming years. So far, about 400 IKK's have been selected for implementation; a considerable number of these will be realized with Netherlands aid funds.

A task force, consisting of staff of the Indonesian Public Works Ministry (Directorate General of Cipta Karya), and of Netherlands consultants, developed a special, standardized approach, in order to provide people with safe water in the shortest time possible and at minimum costs.

The main characteristics of this approach are:

- low cost water supply systems, resulting from a relatively short design period, restricted availability of water (both litres per head per day and overall availability in litres per minute per connection) and decentralized storage (in the homes and at public taps)
- standardized survey, design and construction procedures that allow the bulk of the work to be done at district or regional level, and - as far as possible - without having to refer to the Ministry for expert advice
- manuals for survey, design, supervision, operation and maintenance, and for community involvement related to water supply
- centralized procurement and stockpiling of pipes, accessories, pumps, generators and other mechanical/electrical water supply system components
- application of standard package plants for surface water treatment
- testing and monitoring programmes
- setting up an institutional management system for water enterprises, and related training programmes.

LARGE-SCALE IMPLEMENTATION OF WATER SUPPLY SCHEMES FOR SUB-DISTRICT CAPITALS (IKK) IN INDONESIA; A STANDARDIZED APPROACH

1. INTRODUCTION

The Indonesian archipelago consists of more than 13,500 islands, with a total land area of almost 2 billion square kilometres. Its administrative structure comprises 27 provinces. These are divided into 247 regencies or districts (Kabupaten), which in turn are subdivided into some 3,450 sub-districts (Kecamatan).
There are 64 self-governing cities and towns (Kotamadya), approx. 3,000 other towns and more than 60,000 villages.

Indonesia has a total population of some 155 million, of which about 124 million (80 percent) live in rural areas.

At the end of the third five-year development plan (Repelita III, which ended early 1984) approximately 40 percent of the urban population was provided with water from piped water supply systems. Of the rural population only about 32 percent was supplied with drinking water from reliable supplies.

The targets set by the current development plan (Repelita IV, 1984/85 - 1988/89) are as follows:

- urban population: 75 percent coverage (on the basis of the 1990 population);
- rural population: 55 percent coverage.

2. GUIDELINES FOR WATER SUPPLY DEVELOPMENT IN INDONESIA

During earlier development plans the so-called "Basic Needs Approach " (BNA) was taken for urban areas in Indonesia; these being defined as all towns with a population of at least 20,000.
Schemes under this programme were based on an average domestic supply level of 60 lpcd (litres per capita, per day), with a provision of 25 percent for non-domestic water demand. Half the population covered by a scheme was served by private connections (house or yard connections, HC/YC) and half by public standposts (PS).

For towns with a population smaller than 20,000 the so-called IKK approach was followed.
IKK is an abbreviation of Ibu Kota Kecamatan or: Sub-District Capital. Not all sub-district capitals are included in the IKK programme, however; the larger towns are covered by the BNA programme and sub-district capitals with a population smaller than 3,000 are considered rural.
On the other hand non-IKK towns with a population between 3,000 and 20,000 are included in the programme.

Since the beginning of the fourth development plan, Repelita IV, the earlier rather rigid BNA criteria have been modified, and at present five categories of towns are identified, each with its own service level:

WATER SUPPLY LEVELS FOR REPELITA IV

Classification	Population size	Average domestic service level	Number
Metropolitan city	over 1,000,000	120 lpcd	7
Large city	500,000 - 1,000,000	100 lpcd	5
Medium town	100,000 - 500,000	90 lpcd	41
Small town	20,000 - 100,000	60 lpcd	236
IKK town	3,000 - 20,000	45 lpcd	3,137
TOTAL			3,426

As before, half the population is to be served by HC/YC and half by PS. This is now related to the urban supplies as a group, however, and the ratio may differ from town to town, depending on the local situation.

3. IKK PROGRAMME

The table above shows that more than 3,000 small towns will be covered under the IKK programme. As these towns constitute extremely small urban areas, there are a number of typical constraints regarding funds, trained manpower and institutional development.

In order to overcome these constraints an innovative, and in some respects untried, method of designs and implementation has been set up by a task force consisting of the Indonesian Ministry of Public Works (Directorate General of Cipta Karya), the Australian Advisory Team and Dutch Consultants. Major characteristics of this method are:

a. the cost of the distribution system, traditionally the most expensive part of each water supply scheme, is minimized by virtually eliminating peak flows. This is accomplished by using decentralized storage and flow restrictors, and by operating the schemes for 24 hours per day. Consequently the cost of pumps, generators, etc. is also reduced and no water meters are required;
b. simple and standardized designs are used, for standard system capacities of 2.5, 5 and 10 l/sec;
c. a standardized survey and design procedure has been fixed and laid down in manuals that can be used by local personnel. Only in exceptional cases, and generally when surface water has to be used as water source, is assistance from the Cipta Karya Planning Centre in Jakarta required;
d. pipe materials, pumps, generator sets, fuel tanks, pressure vessels and some other standard equipment are purchased centrally and stockpiled for a larger number of systems.
Standardization of type and quality of materials also allows interchanging of components between systems;
e. construction of deep wells, supply and installation of mechanical/electrical equipment, and standardized package treatment plants are covered by larger contracts, again combining the works for a group of IKK.

3.1. Design criteria

The IKK design criteria are as follows:

DESIGN CRITERIA IKK WATER SUPPLY PROGRAMME

No.	Item	Criteria
1.	Design period: production unit/distribution system	5 years
	transmission pipes	10 years
2.	Supply level : public standposts	30 lpcd
	house connections	60 lpcd
3.	Population served (intermediate target)	75 %
4.	Ratio population served by HC and PS (national average; flexible for individual towns)	1 : 1
5.	Non-domestic water allocation	5 %
6.	Allowance for unaccounted-for water	15 %
7.	Peak factors: maximum day	1.1
	maximum hour on average day	1.0
8.	Design group: public standpost	200 people
	house connection	10 people
9.	Daily supply hours	24 hours
10.	Storage : at public standposts	3500 litres
	at private connections	300 litres

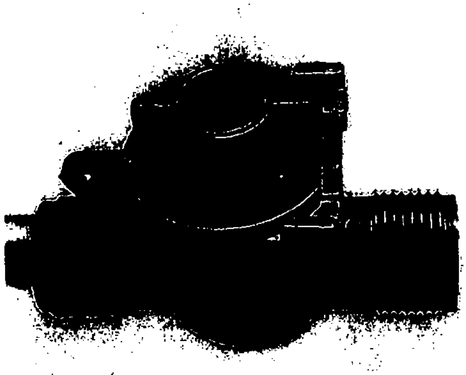
3.2. Decentralized storage

In the IKK approach full use is made of the typical Indonesian habit of having a water reservoir, the "bak mandi", in the bathroom. It is used for storing water, and water is scooped from it for bathing.
It is recommended that for house connections a reservoir of at least 300 litres is used, mainly for washing and bathing, together with a small elevated wall-mounted tank of approx. 20 l for drinking and cooking. Water is provided at a fixed rate of 600 l/day per house connection.

At public standposts storage is provided in tanks with a volume of 3.5 cubic metres. From these a total of 200 people can draw up to 30 litres per head and per day.

3.3. Prevention of peak flows

Even the provision of completely decentralized storage, at all public standposts and house connections, is no guarantee that the flow in the distribution system will be constant over the day. For that reason so-called flow restrictors have been installed in the inflow pipes to all public standposts and house connections. They ensure that, irrespective of the water pressure, the flow to the public standposts is kept almost constant at 4.6 l/min, and at 0.46 l/min to house connections.
The flow restrictors are a crucial element in any IKK scheme as without them the distribution over the distribution area of the limited amount of water available would inevitably lead to inequality of supply. By restricting the flow to a trickle, they ensure a daily supply of 600 litres to each house connection, and 6000 l/day to each public standpost.



Newest, reversible, type of flow restrictor

The flow restrictors currently in use are provided with an insertion restrictor unit that can be reversed, so that any foreign matter can be flushed out. Flow restrictors all have a common body, with a restrictor unit that can be adapted to the required flow rate.

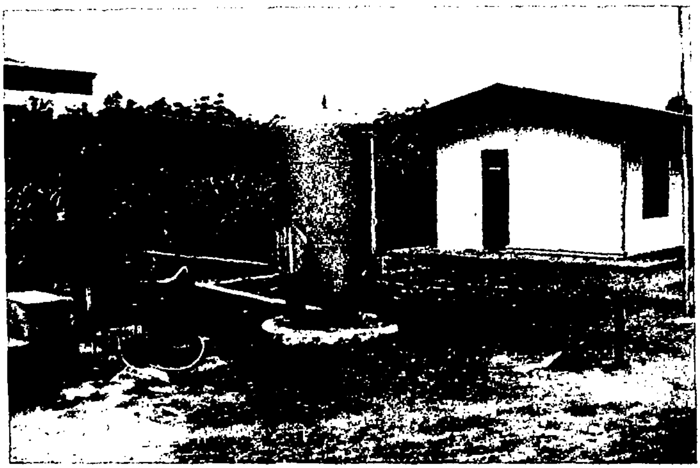
3.4. System design

Standard designs have been prepared for systems using, in order of preference, spring water, shallow groundwater, deep groundwater, lake water and river water, at standard capacities of 2.5, 5 and 10 l/sec.

An IKK Survey and Design Manual, the latest version of which was completed in December 1983, gives guidelines to the provincial staff for carrying out survey and design work for IKK systems, using a modular approach. The main topics covered in the survey section of the manual are: (i) source survey, (ii) town mapping, (iii) population survey; those in the design section: (i) source selection, (ii) selection of treatment, (iii) design of pipe systems, (iv) selection of pumps and power supply and (v) preparation of tender documents.

Selection charts are included for each type of water source and each standard capacity, differentiating between schemes with gravity supply or pumped supply, with or without booster pumping stations (the former in cases where the total head on the source pumps would exceed 100 m), and with or without break-pressure tanks (the former only when static pressures in the pipeline would exceed 80 m).
The selection charts refer to standard drawings of system components with corresponding standard bills of quantities. Thus, in any particular case, only the layout of the water source, and the transmission/distribution system require specific drawings to be prepared, whereas for all other system components the standard drawings and standard bills of quantities can be used.

Surveys are carried out by small teams of Indonesian consultants or provincial staff, and take one or two weeks per IKK. Designs are normally prepared by the same teams, taking an average of two weeks for completion of the design and tender documents.



Typical IKK headworks with chlorination installation and hydrophore vessel

3.5. Construction

For the supply and installation of materials specific approaches have been adopted, such as stockpiling of materials and packaging of construction components. Standardization of the type and quality of materials and equipment allows them to be shifted between systems, and minimizes the required stock of spare parts.

The construction of deep wells, the supply and installation of pumps, generator sets and other mechanical/electrical equipment and of the standardized package treatment plants, are normally combined in larger contracts, covering a number of IKK systems.

By contrast, all civil works, pipe laying works and construction of public standposts are executed by small, local contractors (district level). They often have next to no experience with water supply systems, necessitating a considerable input from supervisory staff.
Construction is carried out in the following order: (1) production unit, (2) transmission and distribution system, including public standposts, (3) testing of the pipe system, (4) trial running and full operation, (5) installation of house connections.

Installation of house connections is done by trained staff from the water enterprise, but only after applicants have installed the required in-house reservoir.

3.6. Operation and maintenance

In the North Sumatra and Aceh provinces, where the authors have been involved in the design and implementation of IKK systems, these are integrated in the Regional Water Enterprise (PDAM), which in turn is supervised by a regional board. During the first years of operation the PDAM is being technically and financially supported by the central government, through its provincial project offices.

PDAM's responsibility regarding IKK involves:
- support in trouble-shooting and repair of equipment
- financial assistance as required, in the early stages
- technical planning for major extensions
- monitoring of over-all performance of the IKK system
- training support

The staff of the IKK itself is kept as small as possible: a senior operator/head of unit, assisted by two to four operators/administrators.

This local staff has the following responsibilities:
- day-to-day operation and maintenance
- repairs to pipelines
- purchase and control of stocks of consumable materials
- billing and collecting of payments
- preparation of budgets
- installation of house connections

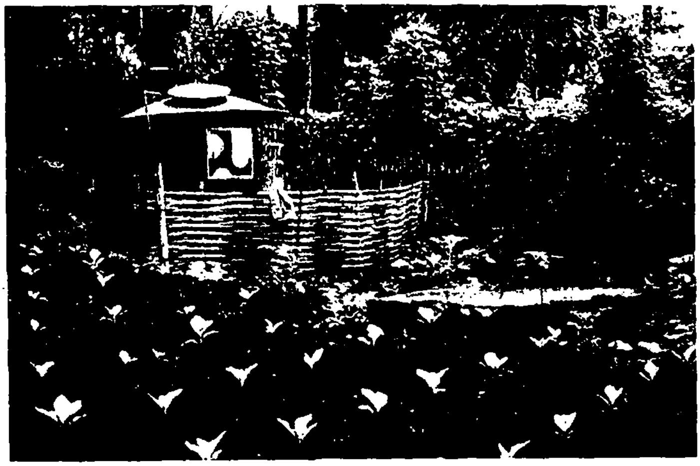
3.7. Cost aspects

Average investment and O&M costs for IKK systems are as follows:

Source type	Module size l/sec	Population served	Construction costs US$	US$/head	O&M costs US$/month
spring	2.5	3,600	54,000	15	295
	5.0	7,200	72,000	10	350
	10.0	14,400	115,200	08	465
deep well	2.5	3,600	100,800	28	642
	5.0	7,200	144,000	20	973
	10.0	14,400	216,000	15	1,524
river water/ treatment	2.5	3,600	118,800	33	1,226
	5.0	7,200	151,200	21	2,016
	10.0	14,400	230,400	16	3,226

The table clearly shows that the use of surface water results in investment costs as well as monthly operation and maintenance costs that are considerably higher than if groundwater from springs or wells is used. The fact that the treatment of surface water also requires a higher level of training of the IKK staff has led to a recommendation not to base IKK water supply systems on surface water sources.

According to present policy, schemes are expected to fully cover their O&M costs after three years. The monthly water rates per household, necessary to recover these costs, are shown in the table below, both for the original 50:50 model and for the 80:20 model (80% of the population supplied through house connections). In both cases it is assumed that the tariff for water from public standposts is 50 percent of that for water from private connections, and that 90 percent of outstanding bills are actually collected.



Typical IKK public standpost

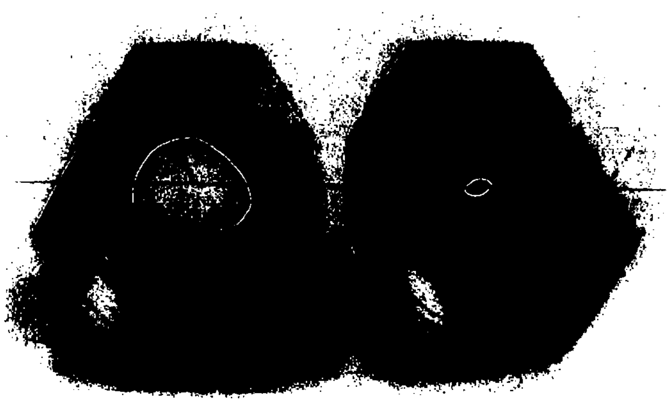
MONTHLY WATER BILLS PER HOUSEHOLD, NECESSARY TO RECOVER O&M COSTS

source type and size	50:50 option house connections No.	monthly rate (US $)	public standposts No.	monthly rate (US $)	80:20 option house connections No.	monthly rate (US $)	public standposts No.	monthly rate (US $)
spring								
2.5 l/s	180	1.46	9	0.36	253	1.22	3	0.31
5.0 l/s	360	0.86	18	0.22	507	0.72	6	0.18
10.0 l/s	720	0.57	36	0.14	1,014	0.40	12	0.12
wells								
2.5 l/s	180	3.17	9	0.79	253	2.66	3	0.67
5.0 l/s	360	2.40	18	0.60	507	2.01	6	0.50
10.0 l/s	720	1.88	36	0.47	1,014	1.58	12	0.39
treatment								
2.5 l/s	180	6.05	9	1.51	253	5.08	3	1.27
5.0 l/s	360	4.98	18	1.24	507	4.17	6	1.04
10.0 l/s	720	3.98	36	1.00	1,014	3.34	12	0.83

In the 50:50 model, monthly water bills would range from $ 0.57 to $ 6.05 for house connections, and from $ 0.14 to $ 1.51 for public standposts. Assuming that most people would not be willing to pay more for water than for electricity, $ 2 per household and per month would be the maximum acceptable water bill. This means that with the 50:50 option only spring sources could be used . Even in the 80:20 option only spring systems and larger well systems result in monthly water bills lower than $ 2.

3.8. Public acceptance

The use of flow restrictors is the unique and indispensable characteristic of any IKK system. It may, however, also prove to be its Achilles' heel. During several evaluations of IKK systems it was found that the flow restrictor can be a source of disappointment and even rejection. Especially the trickle flow resulting from the use of flow restrictors, in a country where water, though often not of the required quality, is generally plentifully available, may contribute to that feeling of disappointment. In several cases, therefore, flow restrictors have been tampered with or removed altogether.



Flow restrictor, original (right)
and after local "modification" (left)

In consequence, appraisals of IKK systems in operation have led to recommendations either to replace the present flow restrictors by others of a larger capacity (with the option of reducing the total number of supply hours, so that the daily allowance per head would remain the same) or to dispense with them altogether. A draw-back of the latter solution would be the tremendous increase in per capita costs, however.

Experience has shown that, irrespective of possible future modifications, efforts to explain the principles of the IKK set-up to the local government officials and the prospective users will have to be stepped up, if the IKK principle is to gain public acceptance.

4. CONCLUSIONS

The IKK water supply concept introduced in Indonesia in 1981 is a novel approach to solving the problem of supplying a large number of semi-urban areas with water, using the typical Indonesian institution of the "bak mandi". The resulting trickle flow apparently results in some disappointment on the part of the public, that seems to expect a more plentiful availability of water. It is not clear, however, whether or not the

total volume of water that is made available per head and per day is considered sufficient.
Based on the experience gained with the IKK systems already in use, a number of modifications that have been proposed to the Indonesian authorities will have to be studied in detail. Meanwhile efforts continue to achieve a sufficient level of acceptance of the system, through a comprehensive community education and training programme.

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APPRAISAL OF RURAL WATER SUPPLY PROJECTS IN INDIA

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ABSTRACT

Financial assistance for a number of rural water supply schemes in India is provided by the Netherlands Government, in the framework of its bilateral assistance programme. Proposals for the schemes submitted for Netherlands financing are prepared by the relevant technical departments of the Indian Government and appraised by teams of experts set up for that specific purpose by the Netherlands Government.

This paper deals with some typical aspects of rural water supply schemes, in the Indian States of Kerala and Uttar Pradesh, that have been appraised by combined missions of DHV staff and Indian experts. Supplementary experience from other Netherlands-financed rural water supply schemes in India is included. Details are provided of the technical and financial aspects of these schemes, their operation and maintenance, and notably the problem of free access for all, in relation to the revenue producing capacity of the projects.

Attention is also paid to health education, drainage and sanitation.

APPRAISAL OF RURAL WATER SUPPLY PROJECTS IN INDIA

1. INTRODUCTION

From 1976 onward the Netherlands Government has been involved in the financing of rural water supply schemes in India, through its bilateral assistance programme. In the framework of this programme, funds are made available to the Indian Central Government, as soft loans or grants, and in turn made available by the Central Government to the relevant State Government.

The Netherlands bilateral assistance programme recognizes both financial and technical assistance (the latter being, for instance, projects carried out in the recipient country by expatriate teams). Water supply programmes in India receive financial assistance only, however. This means that the project proposals, preparation, execution and supervision are entirely in the hands of Indian government institutions and contractors.
In order to check the economic and technical feasibility as well as the progress and final impact of such projects, the Netherlands Government requires them to be appraised, in the proposal stage, and/or during or after execution of the project.

Whereas the Netherlands assistance covers rural water supply projects in the States of Himachal Pradesh, Uttar Pradesh, Andhra Pradesh, Gujarat and Kerala, comprehensive appraisal missions have so far been carried out for Kerala and Uttar Pradesh only. In both cases a combined and multidisciplinary Indo-Dutch mission was sent to the project area. The mission's findings and those of regular progress evaluation missions, on technical, financial, educational, institutional and other aspects of the rural water supply schemes are discussed below.

2. METHODOLOGY OF APPRAISAL

Comprehensive appraisals of rural water supply schemes have been carried out by Indo-Dutch missions that generally included the following categories of experts: water supply expert, planning expert, sociologist, hydro-geologist and/or hydrologist, economist, health education/community participation expert.

Preferably after a pre-appraisal mission had established which persons and departments should be contacted to derive the optimum benefit from the appraisal mission itself, the latter departed to the relevant State capital and started discussions there. These concerned the selection of schemes and of the villages covered by them; technical aspects regarding the design, selection of materials and quality control; supervision of construction; operation and maintenance aspects; institutional aspects (availability of a sufficiently equipped organization and of adequately trained staff for the design, supervision and O&M of the schemes); financial viability of the schemes; financial contribution to O&M by the State Government; etc.
Invariably extensive field visits were paid to the project area, and when necessary State and Central Government policy matters were discussed with the relevant authorities.

Many of these aspects are common practice in any appraisal. Those that specifically apply in the Indian situation are discussed in more detail below.

3. CRITERIA FOR THE SELECTION OF SCHEMES

Government policies

It appears that there are often discrepancies between the policies of the Governments involved, which in this case may be compounded by differences between Central and State Government policies in India itself.
Whereas the Netherlands Government pursues a policy whereby attention is paid first and foremost to improving the living conditions of the socially weaker sections of the community, local interests may favour an "area-coverage" approach.
In such an approach all villages in a certain District or part thereof are included in a water supply scheme, regardless of priorities regarding water demand. Then, although the Netherlands Government may favour a "worst first" approach: financing rural water supply schemes consisting of "problem villages" or "scarcity villages" only, it has to be satisfied with the selection of those schemes that show a relatively high percentage of such villages.

Traditional open village well

Scarcity or problem villages

According to the official Indian standards, "scarcity" or "problem" villages are villages where at least one of the following applies:

- the depth to the water table is more than 15 m
- the source dries up in summer
- the water is oily, brackish, or contains excess fluorides (more than 1 mg/l)
- the distance to the source exceeds 1.6 km (1 mile)

In its efforts to meet the targets of the International Water Supply and Sanitation Decade, the Indian Government faces the problem that in 1972 a list of scarcity villages was drawn up, which later proved to be less than complete. Applying the criteria mentioned above, would in practice result in an annually growing number of scarcity villages, so that - at least on paper - the Indian Government would seem to get farther and farther away from the Decade target. For political reasons the Government of India therefore restricts itself in its present planning to those villages that are included in the 1972 list.

Public standposts or financial viability?

A further difference in approach emerges in the provision of public standposts. Whereas in the Netherlands' vision public standposts are the logical means to provide the weaker sections of the population with water, requiring large numbers of these standposts to be included in the schemes, the local technical staff repeatedly argue that such large numbers of public standposts would take away any incentive to apply for - more expensive - private connections. As the latter generally constitute the only important means of income for a water supply undertaking, such an action would jeopardize the financial viability of the operation and maintenance of the schemes.
This subject is discussed in more detail under item 6 below.

4. WATER DEMAND

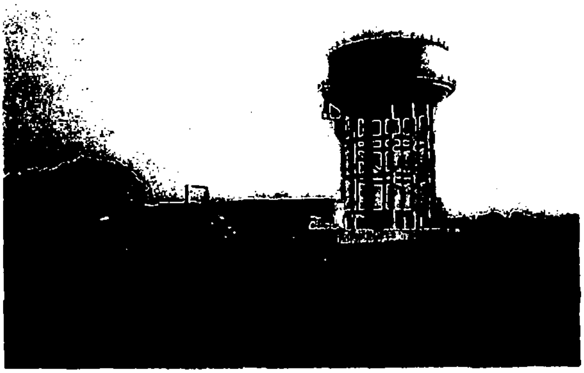
Per capita water demand

Water demand is generally determined on the basis of the "Manual on Water Supply and Treatment" from the Ministry of Works and Housing in New Delhi. For rural water supply this manual recommends a minimum of 40 lpcd (litres per capita per day) in case water is supplied by public standposts, and at least 70 lpcd where private connections are contemplated. In some States a further distinction is made between villages with a population smaller or larger than 4,000 (at the end of the design period), with 90 lpcd taken for the larger villages.

In recent years, in its efforts to meet the Decade targets within the limited time and budget available, the Central Government has proposed to lower the per capita water demand criterion for rural piped water supply schemes to 40 lpcd. The consequences, mainly regarding the financial viability of operation and maintenance, as discussed under item 6 below, led the Netherlands Government to withhold approval for such schemes, after which the original design parameters were reinstated.

Population projections

Although the Manual on Water Supply and Treatment distinguishes a number of methods to determine the design population (i.e. the population after 30 years), normally a population growth of 50 percent over a 30-year period has been taken as the standard. This represents an annual growth rate of only 1.36 percent, which is far lower than the results of the 1981 census (available only recently) indicate. The much higher growth rate over the decade 1971 - 1981 is now taken as the basis of population projections. Because of its large influence on the design capacities of the schemes, and because it is not clear whether this growth rate will continue for some decades to come, there is a tendency to reduce the design period (traditionally 30 years) to 20 - 25 years. A reduction to 15 years, as recently proposed in combination with the earlier mentioned 40 lpcd water demand criterion, has so far not gained acceptance.



Headworks of typical Uttar Pradesh water supply scheme

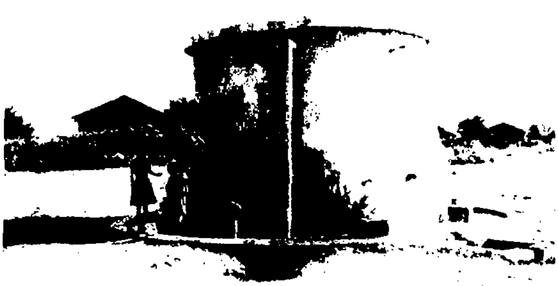
5. TECHNICAL ASPECTS

Types of schemes

A wide variety of rural water supply schemes is included in the Netherlands assistance programmes. On the one hand there are regional schemes that supply up to 100 villages at a time and are based on groundwater (Gujarat) or irrigation water (Andhra Pradesh), on the other hand there are single-village schemes and hand pump schemes. In the State of Uttar Pradesh most schemes have a strictly standardized approach: they each cover a group of villages and are based on two deep wells that pump water into an elevated reservoir. From there it is distributed by gravity, through a simple, branched distribution system. No water treatment is provided, except for safety chlorination at the deep well site.

Types and situation of outlets

Differences in policy often manifest themselves in the way in which the water is made available to the rural population.
Some States limit the type of water outlet to public standposts, e.g. Gujarat State. There water is provided through ground-level reservoirs, so-called "cisterns", to which a series of public taps are connected.



Water "cistern" in Gujarat

The pressure in the distribution system can thus be limited and peak factors reduced. This allows pipes of a lower pressure class and smaller diameter to be used, so that a maximum number of people can be supplied with water for the available funds.
An inherent draw-back of this system is, however, that any modification of the system to make private connections possible, requires a disproportionately high extra investment. As will be discussed later, this type of system may thus be heavily dependent on government financing for its operation and maintenance.

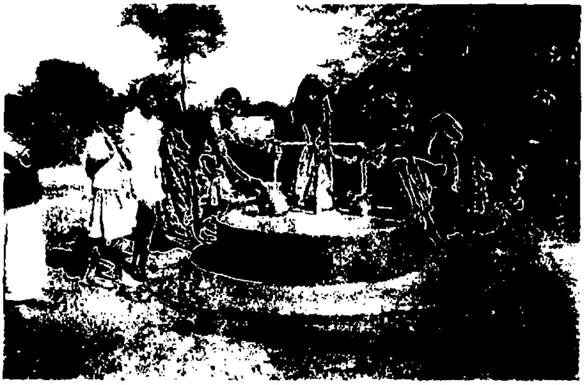
In other States, e.g. Uttar Pradesh, great emphasis is laid on the recovery of the operation and maintenance costs. In practice this means that the number of private connections should be as large as possible, as revenues from public standposts are negligible, and in many cases nil.
As mentioned before, in its financial assistance projects the Netherlands Government emphasizes the supply of water to the poorest sections of the population. This implies that attention should be focussed on public standposts rather than private connections; a course which could jeopardize the financial viability of the operation and maintenance of the schemes.
In practice, therefore, an approach is sometimes followed whereby initially only a limited number of standposts is installed and applications for private connections stimulated. After the demand for private connections has been saturated, the number of public standposts is then increased.

Public standposts

Standards for the number and location of public standposts do not exist. A number of 250 consumers per public tap is sometimes used, whereby one public standpost may have up to 12 taps and more, but generally 1, 2 or 4 taps.
In practice it may not be the number of consumers that is the overriding factor in determining the number of public standposts, but their accessibility, both in terms of distance and "social" accessibility. Sometimes a maximum distance of 250 m is adopted, but fixed standards do not exist.
A working group of the Ministry of Works and Housing is preparing national guidelines on this subject, however.

In India the caste system, and especially the fact that a certain group of the population used to be considered "untouchable" (currently called the "harijans" or "scheduled castes"), has resulted in situations where access to water sources was denied to specific social groups, and where water sources allocated to these groups were intentionally polluted.

In its efforts to eradicate this kind of undesirable situation, the Indian Government has adopted positive discrimination for these "scheduled castes and scheduled tribes".



Public standpost with 4 taps

For rural water supply this means that the first public standpost in any scheme is supposed to be sited in the harijan quarters. Although undoubtedly decided with the best intentions, this rule may backfire, in that other, more affluent, population groups resent the preferential treatment of the harijans, and damage or destroy the public standpost or deny the harijans access to it; a situation that has been reported in some cases.

It may also prove to be virtually impossible for the technical field staff to implement this instruction. Not only will the village leaders expect that water is first made available in the immediate neighbourhood of their houses, but the technical field staff might even suffer bodily harm if they were to install standposts in the harijan quarters only.

The issue is further complicated by the fact that, contrary to popular belief, in practice the rift is not between the socially weaker groups (Scheduled Castes/Tribes) and higher casts. Often it is groups that are als socially weak like the shoemakers or hunters, that do not allow the sweepers to touch their wells, while they themselves do have access to the wells of the higher castes.

In practice at least some of the problems are overcome by providing both the harijan group and the remaining population with a public standpost each. Other standposts, planned to be located in the socially weaker sections may also be erected without further delay, as they will not normally be used by the more affluent people and thus not constitute any threat to reaching the desired numbers of private connections.

6. COST ASPECTS

The effect of design criteria on project feasibility

In order to meet the Decade targets with the financial means available, the Indian Government has sought ways to reduce the per capita cost of rural water supply schemes. Recently it therefore suggested that the design period for rural water supply schemes could be brought back from the original 30 years, to 15 years, whereas the per capita water demand could be put at 40 or 50 lpcd, rather than 70 or 90 lpcd as mentioned in the Manual on Water Supply and Treatment.

The consequences of these modifications were studied in detail for the State of Uttar Pradesh by a combined Indo-Dutch appraisal mission. It compared investment and operation and maintenance costs for three types of schemes:

a. designed in accordance with the traditional design criteria (70 lpcd, 30 years design period);
b. designed on the basis of 50 lpcd and a 30-year design period;
c. designed on the basis of a 40 lpcd initial design, to be expanded to 70 lpcd after 15 years.

In the last case, the final situation would thus be the same as with the traditional set of criteria, but with a first stage of smaller capacity.

For all three options the present value of capital and replacement costs was calculated for a typical scheme, as was the shortfall in revenues for operation and maintenance. It was assumed that over-all water demand at public standposts is 40 lpcd, so that in option (c) no private connections are possible at all in the first stage.
In option (b) a maximum of 17 percent of the population could be supplied through private connections, whereas the traditional set-up of option (a) allows a maximum of 50 percent of the population to be provided with private connections, at a per capita water demand of 100 lpcd.

The present value of capital investment, replacements and O & M subsidies, discounted at 8 percent per annum, can be summarized as follows (related to the population after 30 years):

Type of scheme	40/70 lpcd	50 lpcd	70 lpcd
Capital + replacement cost (US$/head)	13.54	13.36	14.20
O & M subsidy (30 years) (US$/head)	2.69	2.31	1.40
PV of total costs (US$/head)	16.23	15.67	15.60

The 70 lpcd scheme emerges as the least-cost solution, even though the differences in present value between the various options are small. With an estimated total of 7 million people to be provided with piped water supplies and 16.7 million with hand pumps, in the State of Uttar Pradesh alone, it is clear that even very small differences count, however.

The State contributes O & M funds from non-plan moneys, so that in practice it is not possible to plan allocations for the coming years. Even the most optimistic authorities can only express cautious hope that these allocations can be increased proportionally in the coming years. With irrigation and power supply generally coming first on the list of priorities, and present O & M budgets already being far from adequate, it is only realistic to assume that the allocations for O & M of rural water supply schemes will not keep pace with the rapidly growing needs, as more and more new schemes are commissioned.

Construction of 40 lpcd schemes, which imply no income from private connections, should not be undertaken, since even sub-standard O & M can not be guaranteed. The Netherlands Government thus withheld approval for financing of schemes set up according to the revised criteria until the original criteria were re-adopted.

70 lpcd schemes have higher construction costs, as shown above, but often these costs are financed with external assistance. O & M subsidies required are much lower, and decrease with time. They offer, moreover, much better prospects for adaptations: if the price of water were to be raised from Rs 0.54 (US$ 0.045) to Rs 80 (US$ 0.067) per cubic metre, no subsidies would be required at all for O & M. While this option at present constitutes a major political problem, the State may have to resort to it in the future.

Financial resources of the State and expenditure on water supply

In India most public services/infrastructure facilities are provided at nominal rates, or even totally free of charge. Rural water supply is an obvious example in this context. Even people who can afford to pay more, get the water free from a hand pump or public standpost or, far below actual cost from a private connection.

At present, some 15 to 20 percent of the State budgets are already spent on Social and Community Services. This is about the maximum that can reasonably be expected to be spent on this sector, which comprises education, research, health, water supply and sanitation, housing, urban development and various welfare programmes. Water supply and sanitation are already the largest single component in this sector, and it is doubtful whether a further increase of expenditures for water supply - necessary to meet the Decade targets - would be acceptable.

Expenditure plans of the States show a growing imbalance between the rising cost of social and other services that are not directly productive, and the proceeds derived by the States from these services. The implementation of many more rural water supply projects will further increase the burden of social expenditure on the State budget, to the detriment of the more productive investments.

This does not even take into consideration the fact that additional capital outlay would be required for sanitation and drainage. The importance of these facilities will grow during the coming years, as the more ample availability of water, both in the urban and rural areas, will render measures here increasingly necessary.

Emphasis must therefore be laid on a technical set-up and water tariff system that allows a major part of investment, replacement and O & M costs to be recovered within a reasonable period. This means that short-term solutions, such as reducing the availability of water per capita, should not be pursued. It also means that large numbers of public standpoints only seemingly contribute to the benefit of the weaker sections of the community; the probably ensuing lack of sufficient operation and maintenance funds is more than likely to jeopardize the supply of water, especially to this population group.

7. COMMUNITY ACCEPTANCE

The importance of an assured access to the water outlets by all sections of the community has been discussed in some detail before. The siting of public standposts is, therefore, of great importance.
Perhaps the participation of the people in decision-making is even more important in this respect. It is essential that acceptable and feasible procedures are adopted, e.g. making use of community-based water commit-

tees, with assistance to the State Government being provided by socio-economic units. These could, for instance, comprise a sociologist, economist, aerial photograph interpreter or map specialist and a health education specialist, and operate at District level.

In Gujarat State, positive results have been obtained from an awareness campaign in which the importance of clean water and of an effectively functioning water supply system were the subjects of mime theatre performances in the project villages.
A health education component in rural water supply projects is a prerequisite for deriving the optimum benefit from the improved availability and quality of water. The organization of such a component could also be the responsibility of the socio-economic units. In addition, these units would be expected to increase the awareness of people that clean water is a scarce resource that must not be wasted.

8. LINKED ACTIVITIES

The limited length of this paper does not allow covering a number of other subjects that are equally important for a successful rural water supply project.

Operation and maintenance are, of course, of crucial importance; in particular guaranteeing uninterrupted availability of power may be a problem in a country where vast amounts of power are used for irrigation purposes. In various projects it has, therefore, been necessary to make available additional funds for erecting so-called exempted feeders to the schemes' headworks, i.e. power lines that are not subject to the frequent power rostering the State Electricity Boards are often forced to impose in the dry season. In others back-up generating sets have been included to cover any powerless period.

The training of staff for the schemes is an equally important aspect that receives growing attention in the various States. In-house training facilities exist in several States; in others these are being provided. In Gujarat State, for instance, a comprehensive staff training programme has recently started with World Bank assistance, including the construction of own training facilities.

An aspect that has received insufficient attention so far, is sanitation and drainage. The high population densities, the absence of sanitation facilities in most villages, and the often very poor drainage facilities underline the urgency of this problem in the project areas. Since water supply projects can only have a negative effect on the drainage/sanitation problem, all of these projects should include a drainage/sanitation component.
Experience in this field is limited, however. For that reason it has been proposed to carry out pilot projects, in combination with the implementation of rural water supply schemes. Based on the experience gained, proposals could be drafted for the inclusion of a comprehensive sanitation/drainage component in all such schemes.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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GENERAL WATER RESOURCES ASSESSMENT FOR RURAL AREAS

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ABSTRACT

General water resources assessment for rural areas requires information on hydrologic attributes of interest and their statistical characteristics for feasibility studies and for the design and operation of water projects for specific purposes. The statistical characteristics, usually determinable at gaging stations, can be analyzed to delineate hydrologically homogeneous regions and to develop significant relationships between these characteristics and measurable watershed factors for each region. Water resources assessment may be needed for the design of ground-water wells, in-channel and off-channel reservoirs, and farm ponds; for the determination of protected streamflow, and for management of effluent discharges.

Hydrologic attributes and their statistics were developed for the state of Illinois, USA, by dividing it into 10 hydrologically homogeneous regions. The attributes and their statistics were highly correlated with drainage area and sometimes also with main channel length and/or slope. General water resources assessment for rural areas can be made with the help of regional equations. A new interactive basinwide water resources assessment model has been developed to modify the hydrologic attributes downstream of any new water resources development.

Keywords: water resources assessment, regional analyses, storage reservoirs, farm ponds, protected streamflow, regression analyses, irrigation, water supply.

INTRODUCTION

Dependable, adequate, and good quality water supply is needed for the welfare and health of rural communities and their livestock. Availability of water for irrigation, at prices justified by the increase in crop yields, reduces the uncertainty of the yields during extended dry or drought periods. Even availability of lesser quantities of water is helpful in supporting supplemental irrigation to reduce crop stress during crop development periods and thus save farmers from severe economic losses. Various types of water resource developments are possible in rural areas, such as ground-water wells, in-channel and off-channel reservoirs, and farm ponds. The suitability of one or a mix of alternatives depends on the water resource assessment for the area under consideration, any impacts on the area downstream, and economics of benefits and costs. Information on the hydrologic attributes necessary for designing a water resource development, and their statistical characteristics, is relevant to the feasibility study and to choosing the most economical alternatives. Some of the purposes for which water resource assessments are needed are given below, followed by an example of a regional study which can provide the relevant hydrologic design parameters.

GROUND-WATER WELLS

The extensive ground-water aquifers which can support pumpage for various uses, without exceeding their firm yield, require careful investigation because they can provide independent and moderate cost water supplies for rural areas with low population density. A major cost in ground-water supply is the need for long conveyance pipelines. Distributed development of ground water can be planned to minimize this component of cost. Another important economic factor is the depth to the ground-water pumping level because this is directly related to the capacity of the pumping equipment and energy costs. The availability of adequate capacity shallow aquifers (or aquifers at a depth of 200 feet or less) has the potential of making irrigation economical from ground-water wells. In the case of much deeper aquifers, the water supply for humans and livestock will still usually be cheaper than from other sources.

Ground-water quality is usually much better than surface water quality in terms of suspended solids, contaminants, pathogens, etc. In many cases, no special treatment is needed except chlorination and/or aeration to precipitate soluble iron compounds. To ensure a continuous supply, more wells are drilled than needed to serve as standbys in case of breakdowns or repairs. There is extensive literature on aquifer testing and yield investigation, selection of well types, well drilling, installation of pumping equipment, and proper operation of the wells.

IN-CHANNEL STORAGE RESERVOIRS

For the economical design of an in-channel reservoir, information is needed on the streamflow variability with time, the pattern and magnitude of various uses, and the cost functions for conducting economic analysis. The streamflow record may be a daily/weekly/monthly flow sequence, historical or generated from the data available from nearby gaging stations, covering a period of 30 years or more. The flow sequence is analyzed to determine

1) the magnitude and frequency of various-duration drought flows as well as the months of their occurrence; 2) the low and high flows of 1-, 3-, 7-, 15-, and 31-day durations, corresponding to 2-, 5-, 10-, 25-, and 50-year recurrence intervals; and 3) the 2-, 10-, 25-, 50-, and 100-year flood estimates. Values of needed reservoir storage in inches over the watershed to meet various water withdrawal rates and recurrence intervals are developed for the critical drought duration associated with a water withdrawal rate and drought frequency (Terstriep et al., 1982). As an example, these are given in Table 1 for the Horse Creek at Pawnee, Illinois, drainage area 52.2 sq miles. In this table T is the recurrence interval in years, S is the needed storage in inches of water over the watershed, and D is the critical drawdown or drought duration in months. The numbers 2, 5, ..., 100 denote water withdrawal rate in percent of mean flow.

Table 1. Storage capacity in inches (S) and critical drawdown duration (D)

T	S/D	2	5	10	15	20	30	40	60	80	100
5	S	0.06	0.14	0.34	0.55	0.75	1.27	1.82	3.10	5.02	7.45
5	D	4	4	6	6	6	8	8	14	14	22
10	S	0.10	0.24	0.48	0.77	1.08	1.74	2.43	4.87	8.72	12.57
10	D	7	7	7	9	9	10	10	28	28	28
25	S	0.11	0.27	0.72	1.34	1.96	4.03	6.73	14.40	22.10	29.80
25	D	8	8	18	18	18	32	46	56	56	56
50	S	0.22	0.64	1.53	3.44	5.36	9.28	13.27	21.24	29.22	37.20
50	D	20	20	54	56	56	58	58	58	58	58

Net evaporation (evaporation minus precipitation) loss in inches from the reservoir for various durations and frequencies of drought is estimated from the regional information on precipitation and lake evaporation (Roberts and Stall, 1967). The loss can be interpolated for the critical duration and drought frequency for the desired rate of water withdrawal. The reservoir storage is reduced with time because of the sediment retained in the reservoir. The loss in storage due to sedimentation can be estimated from the annual sediment load and the trap efficiency (Brune, 1953). The initial storage capacity, then, equals the sum of 1) net storage needed to permit desired water withdrawals throughout the useful life of the project, 2) reservoir storage needed to make up for the net evaporation loss, and 3) reservoir storage needed for sediment accumulation and any dead storage or certain minimal storage that needs to be left in the reservoir to allow continuance of other uses, such as fish habitats and recreation, though at a reduced level.

The obvious uses of a reservoir in a rural setting are: water supply for humans and livestock, water for irrigation during droughts and critical crop development periods, and water-based recreation. Because of the rather low population density in rural areas, the per unit cost of water supply is high and usually the per capita use is somewhat less than in urban areas. The water needs to be properly treated to conform to applicable standards and supplied to homesteads through a pressure conveyance system consisting of pipes. Water for irrigation may be withdrawn directly from the reservoir or routed through the reservoir and pumped from the channel downstream. Whereas the water supply for human and livestock use does not vary much over the year, the water demand for irrigation is not only seasonal but also greatly variable depending on the weather during the crop season. The period of water-based recreation may vary from a few months to the whole year depending on the regional climate extremes in various seasons.

In-channel reservoirs suffer from loss of storage due to entrapment of sediments carried with the inflowing waters. The problem is severe for reservoirs with low capacity-inflow ratios because they may use only a small portion of annual inflow but trap 70 to 90 percent of annual sediment. The dredging of sediments is very costly and requires suitable disposal sites within a reasonable distance from the reservoir. The effective method for significant reduction in sediment inflow is the introduction of soil conservation measures in areas delivering most of the sediment to the stream. Another method for small reservoirs is the provision of sufficient-capacity sluices near the bottom of the impounding dam to pass high flows carrying most of the sediment; this reduces sedimentation in the reservoir and improves reservoir water quality because of the flushing out of nutrients with the sediments for high flows routed through the sluices. The capacity of the sluices and main spillway is governed by the usual high flows and the design flood, respectively. The design flood frequency depends on the reservoir costs and on relative potential for losses from higher floods than the design flood, during the useful life of the project. A major factor is the potential for loss of human life.

Some mandatory low flow releases may be necessary to provide some flow downstream of the reservoir during droughts when reservoir levels are below the normal pool. Provision of such releases mitigates, to some extent, the adverse impacts of drought on aquatic life and river ecology for a certain reach of the river downstream of the dam. This, however, increases the design storage and hence the cost of the reservoir. Singh (1982) provides information on incremental reservoir costs for various low-flow release levels and associated improvement as shown by preference of the fish for the aquatic habitat.

The cost for the construction of a reservoir and appurtenant works can be estimated by using standard procedures. Similarly, the annual OMR (operation, maintenance and repair) cost can be estimated. The benefit-cost analyses not only indicate the suitable size of the development but also provide costs of meeting water requirements from in-channel reservoirs, for comparison with other feasible alternative(s).

OFF-CHANNEL STORAGE RESERVOIRS

Cost of constructing an in-channel reservoir and its OMR may sometimes be so high sometimes that irrigation is not an economic feasibility. There may be other factors that make such a development undesirable, such as a very small capacity-inflow ratio but a very high reservoir cost per unit of water needed, nonavailability of a suitable site for dam construction, general low landslopes causing submergence of large areas of productive land under normal reservoir pool level, highly significant adverse impacts downstream in terms of ecology and environment, and potential for considerable loss of life in densely populated areas downstream in the event of dam failure. Under such conditions, an off-channel or a side-channel reservoir may be economical and practical for meeting water demands for domestic and livestock needs.

An off-channel storage reservoir is constructed close to the stream and at a ground level higher than river water level during rare flood events. A natural depression is preferable to reduce cost; otherwise the required storage capacity is created by excavation and an earthen embankment is made around the excavated area with the excavated material. In order to control seepage losses and growth of aquatic weeds, the sides and bottom of the reservoir may be lined with vinyl sheets or other suitable lining. The design reservoir capacity should be such that the water supply needs can be met from approximately 80 to 90 percent of the storage during a design drought of a 25- or 50-year recurrence interval, for the durations when water is not available for withdrawal from the river or when the withdrawals are not sufficient to reverse the lowering of water level in the reservoir. The overall depth of the reservoir is determined to provide the needed storage with a freeboard of 3 ft or more, as well as to make up the storage loss due to net evaporation for the critical duration of the drought.

Water withdrawals can be made from a stream or river during moderate to high flows or when the flow exceeds the protected streamflow or that needed to maintain instream flow requirements. The amount of storage necessary to meet a particular demand is estimated from pertinent hydrologic information and factors that affect the amount of water pumped into the reservoir. These factors include the time variability of streamflow, seasonal protected or instream flow requirements, and aspects of design and choice of pumping system delivering water to the reservoir. To reduce sedimentation in the reservoir, the pumping system may not be operated when the stream flow is especially turbid. A pumping system which offers a wide and more continuous range of pumping capacities is more efficient in supplying the off-channel reservoir with water during low and moderate flow periods (Knapp, 1982). Use of such a system reduces the amount of storage needed in the reservoir. There is a trade-off involved between the storage design capacity and the flexibility and versatility of the pumping system. The cost of an off-channel reservoir includes the cost of land, excavation and earthen embankment, lining, pumping system, and intake in the stream or the river.

FARM PONDS

Farm ponds are used for providing water for both livestock and supplemental irrigation during critical periods in crop development when considerable soil moisture deficits occur due to continued dry weather or special soil characteristics. To avoid their filling up with soil eroded from the land, the inflow from the land, if any, is made to shed most of the sediment in a grassed buffer strip of land. During periods of high flow, water is pumped from the nearby streams into the ponds, which are also replenished by the direct rainfall over them as well as inflow from the adjoining lands. These ponds can be lined with impermeable membranes or clay to reduce water loss by seepage. To keep the evaporation loss from the pond surface to a minimum, the pond is usually filled with water pumped from a stream so that the period between filling of the pond and its water use for irrigation is as small as possible. These farm ponds can be effectively used for supplemental irrigation during the usual crop stress period. Though it does not increase the crop yields significantly in average wet years, it does greatly reduce the decrease in crop yield that will otherwise occur because of dry conditions during the critical crop period.

Amounts of water needed to substantially reduce the adverse impact of weather during the critical crop period can be estimated for an agricultural field of a given size when the fraction of land under each crop is known. To this can be added any water demand for livestock, and the loss of water due to evaporation over the period between filling of the pond and its use for irrigation. The excavated pond needs to accommodate all this water with a freeboard of about 2 or 3 feet. A water budget can be prepared to show how the required water volume will be made available to the pond. The sources include runoff from the land, precipitation over the pond, and pumpage from the streams. The flow statistics for the nearby streams need to be developed for the months of interest to ensure that suitable water withdrawals can be made without impinging on their protected flow or instream flow requirements.

PROTECTED FLOW AND EFFLUENT DISCHARGE

Modification of river flow resulting from the construction and operation of a dam or impounding structure or the diversion of water for offstream uses can cause water quality and aquatic habitat problems. Certain minimum flows, seasonal or otherwise, need to be specified for maintaining stream water quality and stream ecology. If the streamflow falls below these flows, no withdrawals for offstream uses may be permitted. These flows are defined as protected streamflows. Low flow criteria for fish and wildlife are being investigated in various countries for determining the suitability of various low flow regimens for fish and wildlife. In order to choose a minimum low flow release (during droughts) which keeps the fishery in good condition and at the same time does not greatly increase the cost of the reservoir, the decision maker needs to know the estimated increase in cost of a reservoir that provides such a release over the cost of a reservoir with no release, for a range of low flows. Some investigations on such trade-offs between incremental reservoir costs and incremental aquatic habitat improvement have been reported by Singh and Ramamurthy (1981).

The consideration of protected and instream flows may significantly increase the cost of in-channel and -- to some extent -- off-channel reservoirs in regions with great variability in streamflow, in which the very low streamflow periods coincide with periods of irrigation water demand and some increased water supply requirements. Under such conditions, area-wide irrigation may not be economical though the farm ponds may still support supplemental irrigation for lands near the streams.

The effluents from a rural sewage treatment plant may be discharged to the stream, at least some distance downstream of the point of intake for rural water supply. If the streamflow is not sufficient at times to provide desirable dilution of the effluents, the effluents may be stored in a lagoon to be released later during high flows, or disinfected and sprayed over agricultural lands.

REGIONAL HYDROLOGY AND RESOURCE ASSESSMENT

Water resources assessment for rural areas can be facilitated by the development of highly significant regional relations between various hydrologic attributes and easily measurable basin factors such as drainage area and main channel slope. The variation in hydrologic response due to physiography, soils, and ground-water aquifers during low flow conditions is accounted for by subdivision of the region, if necessary. The developed relations are used in evaluating hydrologic parameters for investigating the potential of water resources in the area under consideration for meeting one or more of the water use demands. Such relations have been developed for the state of Illinois, USA. The state has an area of 55,744 square miles. The results are presented here for the region drained by the Sangamon River, about 5000 square miles, in central Illinois.

The Sangamon River has two major tributaries: Salt Creek, draining about 1800 square miles of the northern basin; and the South Fork, draining about 1000 square miles of the southern basin. The stream entrenchment decreases from north to south, resulting in less sustained flow in the southern part than in the northern part. There is some physiographic variation also. At one time or another, 23 continuous stream gaging stations were operated in the basin. One of these stations has a daily flow record of 70 years and 11 others have 30- to 45-year records. From the daily flow record available at each station, the hydrologic parameters were developed such as mean annual flow and its standard deviation; mean monthly flows; daily flows corresponding to various exceedance probabilities, used in defining the flow-duration curve; drought flows ~~of~~ 5-, 9-, ~~and 13-month~~ duration, occurring once in 10 or 25 years; 2-, 10-, 25-, 50-, and 100-year flood peaks; and 7-, 15-, 31-, 61-, and 91-day low and high flows occurring once in 10 or 25 years. A preliminary stepwise multiple correlation analysis of these parameters with basin factors of drainage area A, main channel length L, and main channel slope S showed that inclusion of L and S increased the correlation coefficient by a negligible amount but lowered the significance level considerably. The greatest improvement in regional relations is achieved by dividing the basin into 3 sub-basins of Salt Creek, the Sangamon River, and the South Fork.

The results of the regression analyses for some parameters are given in Table 2 as an example. The variables VAR or hydrologic parameters in the table are as follows: $\overline{Q}$ is the mean annual flow and $\overline{Q}(s)$ is the standard deviation; Q(99), Q(95), ..., Q(5), and Q(1) are flows corresponding to the cumulative probability of being equaled or exceeded 99, 95, ..., 5, and 1 percent of the time, respectively, and define the flow duration curve; Q(5,10), ..., and Q(13,25) denote the mean flow during a drought of 5-month duration occurring once in ten years, ..., and a drought of 13-month duration occurring once in 25 years; and Q100 refers to the 100-year flood peak. All flow values are in cfs (cubic feet per second). The symbol A is the drainage area in square miles, the adjusted R^2 is the square of correlation coefficient adjusted for the number of independent variables, and S_e is the estimate of standard error in log units. It is evident that regressions are highly significant for mean flow and flow duration with the exception of Q(99). The correlation for drought flows decreases somewhat with an increase in drought duration and its recurrence interval. The decrease is understandable because of the summation of flows during extended dry periods which are affected considerably by the spatial and temporal rainfall variability, as well as by the land uses and human activities.

Table 2. Sub-Regional Regression Parameters
$\log(VAR) = a_i + b \log A$ $(i=1,2,3)$

VAR	Regression Parameters a_1	a_2	a_3	b	Regression Statistics Adj R^2	S_e
$\overline{Q}$	-0.1965	-0.2047	-0.2090	1.0087	0.998	0.0310
$\overline{Q}(s)$	-0.4223	-0.3756	-0.4778	0.9922	0.991	0.0615
Q(99)	-5.1183	-5.5025	-4.4167	1.9488	0.853	0.3172
Q(95)	-3.6001	-4.0998	-3.1499	1.5833	0.936	0.1864
Q(90)	-2.8257	-3.1131	-2.4983	1.3909	0.951	0.1351
Q(80)	-2.2448	-2.3825	-2.0045	1.2864	0.968	0.0974
Q(70)	-1.7144	-1.8431	-1.5770	1.2070	0.983	0.0736
Q(60)	-1.1887	-1.3260	-1.1065	1.1220	0.990	0.0540
Q(50)	-0.7843	-0.9254	-0.7571	1.0640	0.993	0.0434
Q(40)	-0.5583	-0.6833	-0.5453	1.0507	0.995	0.0357
Q(30)	-0.3554	-0.4473	-0.3559	1.0433	0.996	0.0296
Q(20)	-0.0880	-0.1742	-0.1181	1.0222	0.996	0.0304
Q(10)	0.1903	0.1581	0.1424	1.0175	0.993	0.0409
Q(5)	0.4516	0.4686	0.3926	0.9991	0.992	0.0429
Q(1)	1.0833	1.1219	1.0392	0.8964	0.992	0.0373
Q(5,10)	-3.0436	-3.3670	-2.5957	1.4463	0.967	0.1339
Q(9,10)	-1.1771	-1.5866	-1.0298	1.0208	0.951	0.1250
Q(13,10)	-0.5543	-0.8031	-0.5104	0.9521	0.917	0.1438
Q(5,25)	-3.9739	-4.5625	-3.4852	1.6924	0.944	0.2204
Q(9,25)	-2.0348	-2.3668	-1.8045	1.2006	0.943	0.1503
Q(13,25)	-0.7591	-1.2517	-0.6841	0.9268	0.858	0.2153
Q100	2.7969	2.7873	2.8398	0.5528	0.970	0.0442

The stream network in the whole Sangamon basin has been indexed, and various points along the network have been provided with 1) the stream mileage upstream of the confluence of the Sangamon and Illinois Rivers, 2) the drainage areas upstream, and 3) channel slopes and lengths. A total of 178 hydrologic parameters have been developed at each gaging station and other critical points such as those at stream junctions, downstream of reservoirs, and above major withdrawal or return locations. An interactive water resources assessment model has been developed which not only computes the desired hydrologic information along any desired stream reach or location but also indicates the effect on hydrologic parameters below any contemplated change in water use or discharge. This model allows investigation of various options of meeting new water demands and of the related impacts downstream.

CONCLUSIONS

General water resources assessment for rural areas is facilitated by detailed regional analyses to develop significant relations between various hydrologic parameters and basin factors. A major task in such analyses is the demarcation of hydrologically homogeneous regions and subregions within a region. Interactive modeling of basinwide water resources assessment not only provides speedy evaluation of various modes of water resources development for rural areas in a region but also allows perception of any adverse impacts downstream. The development schemes may include ground-water wells, in-channel and off-channel reservoirs, and farm ponds. A modicum of protection for stream water quality, stream ecology, natural habitats, and fishery is provided by defining protected streamflow and by proper management of wastewater effluents to the stream.

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RECENT EXPERIMENTAL AND COMPUTER MODELING STUDIES ON THE HYDRAULIC RAM PUMP

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ABSTRACT

The hydraulic ram pump (hydram) is introduced and explained. Experiments done on locally-made and commercial hydrams are described and the main results discussed. A computerized model for the hydram is outlined and the main results are plotted. A recent hydram workshop in Arusha, Tanzania is described.

Keywords: Pumping, renewable energy technology, hydram, hydraulic ram pumps, rural water supply.

INTRODUCTION

The hydraulic ram pump (hydram) is a renewable energy pumping device that uses the energy in a small drop (> 2m) in a flowing stream to pump some of that water to a height many times that of the original drop. The following diagram shows a typical hydram installation.

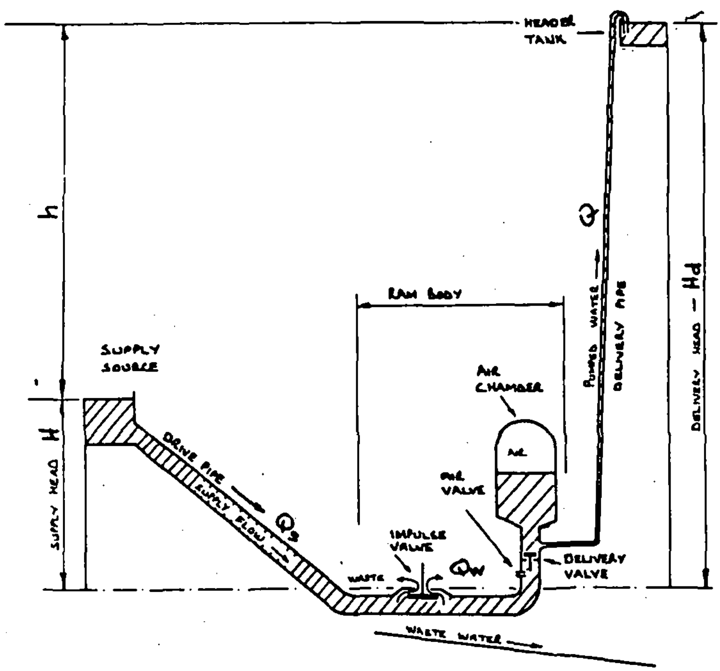

Fig. 1. The Arrangement of a Typical Ram Assembly (Watt, 1978)

OPERATION OF THE HYDRAM

The operating cycle of the hydram begins with water flowing through the drive pipe. Initially the impulse valve (or waste valve) is open and the accelerating flow escapes through this valve. Eventually the pressure and drag forces of the water equal the weight of the valve and it begins to close. Hydrams are designed so that this closing action is as rapid as possible. The valve closes rapidly if it is light and its stroke length is short. However the valve has to have sufficient weight to open quickly later in the cycle and the stroke length and valve opening should be large enough to allow sufficient flow to accelerate and escape rapidly.

The rapidly closing impulse valve creates a water hammer pressure surge in the drive pipe. The more rapid the closure, the higher the pressure created. If the flow in an inelastic pipe is stopped instantaneously, the theoretical maximum pressure rise that can be obtained is $\Delta H = - V\,c/g$

where ΔH = pressure rise (m)
V = the original velocity in the pipe (m/s)
c = the speed of a acoustic wave in the fluid (m/s)
g = acceleration due to gravity (9.8 m/s^2)

When this pressure wave is created, some of the high pressure water can pass by the delivery valve. When this occurs there is a rapid collapse of the pressure surge in the drive pipe.

Three significant things occur when the pressure wave collapses in the drive pipe. Firstly, the delivery valve closes thus ending the pressure surge that is sent to the delivery pipe. The air chamber cushions the pressure surges so that a reasonably continuous flow is sent through the delivery pipe. In this cushioning process the air-water interface is continually agitated and moving. This tends to dissolve the air into the water. The air supply is replenished by a second phenomenon that occurs at this time. A slight negative pressure "recoil" pulse enables air to be sucked into the air valve. Later in the delivery phase this air passes by the delivery valve to the air chamber. This air valve can be a one-way air valve or it can be a very small drilled hole (<1 mm) which releases water during the pressure surge and sucks in air during the collapse of the pressure wave.

The third event that occurs at the end of the pressure pumping phase is that the waste valve opens, either by the action of its own weight or an activating spring. When this happens the flow in the drive pipe begins again. The hydram cycle thus repeats itself continually, at a frequency between 40-200 beats per minute. The fact that this pump operates 24 hours per day with only minimal maintenance is one of its main advantages, especially in developing country situations.

EXPERIMENTAL TESTS

To investigate the potential use of the hydram in developing country situations, a series of experimental tests were conducted in the hydraulics laboratory of the Civil Engineering Department of the University of Ottawa. The aims of these tests were to:

1) Determine the operating characteristics of commercially available hydrams.
2) Compare these results with the operating characteristics for a locally made hydram, which was a revised version of an ITDG hydram (Watt, 1978).
3) Investigate the effect of modifying some of the hydram components with a view to improving the design.

The tests were confined to the smaller models of existing hydrams using a driving head of 1-2 m. The application was thus for small scale applications for isolated small communities drawing from a small stream source. A sketch of the experimental apparatus is given below.

Hydram characteristics are represented by efficiency curves and dimensionless head-flow curves. Tests were made on the locally-made hydram as well as hydrams made commercially by Blake, Rife, Davey and Fleming (Kahangire, 1984). The results for a given site of H = 2 m is given below. Note that these curves are specific for this supply head only. At another supply head

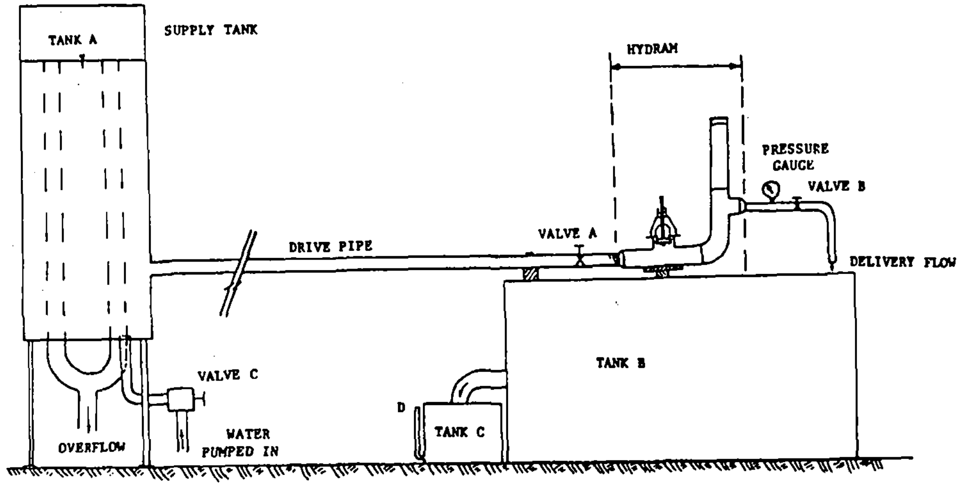

Fig. 2. Apparatus and Experimental Set-up

the efficiency curves will be different. In these curves efficiency is defined as

$$e = \frac{Qh}{Q_w H}$$

where the units are defined in Fig. 1.

Another useful hydram characteristic is plot of head ratio, h/H vs. flow ratio, Q/Q_w. In general these curves are similar in shape, although the larger hydrams like the Blake and the Rife which are designed for a head >2m do not have such good curves as can be noted in Fig. 4.

Detailed tests on various hydram components are described elsewhere (Kahangire, 1978; Schiller and Kahangire, 1984a). The main conclusions derived from these tests can be summarized as follows:

1. A good design of the waste valve is essential for efficient hydram operation. High friction losses through the waste valve are undesirable.

2. Depending on the design of the waste valve assembly, a hydram will operate with maximum efficiency even with long stroke lengths. Increasing waste valve stroke length has the same effect as increasing its weight. Increasing stroke length, however, gives better and smoother hydram operation and higher delivery flows compared to increasing the valve weight.

3. The flow area through the waste valve can be much smaller than the area of the drive pipe with no significant effect on the operating characteristics of the hydram. There is a wide range of waste valve orifice and valve diameters and their combinations with which the hydram will operate normally.

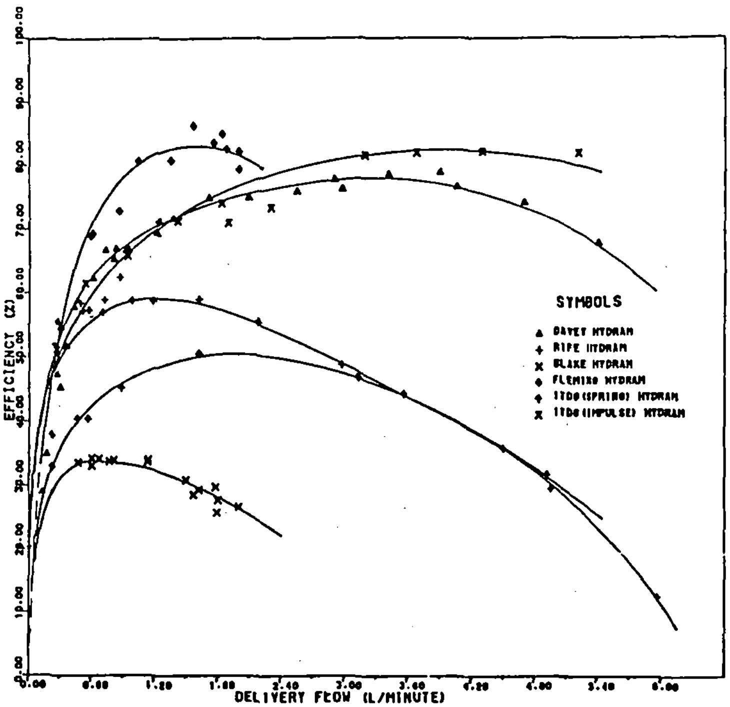

Fig. 3. Efficiency vs. Delivery Flow for Various Hydrams, H=2m

4. V_0, the velocity to initiate waste valve closure, is the most important parameter in hydram operation. This value is generally a function of waste valve design and adjustment. Increasing V_0 reduces hydram efficiency, but increases its pumping capacity and power delivered.

5. The delivery valve should be well designed with minimum friction head losses and resistance to the flow. The grade and hardness of the rubber used should be able to withstand the back pressure from the delivery pipe and air chamber.

6. Increase in hydram capacity, and valve beat frequency is proportional to increase in the supply head.

7. For a given hydram design, the head ratio-flow ratio characteristic curve describes the hydram operation for the given installation and is not affected by changes in the valve stroke length or weight. It is not suitable for comparison of different hydrams except if their drive flow requirements are comparable or about the same. Better hydram operation is obtained with head ratios between 4 and 8. The curves can be used to select suitable hydrams for installations in which the water source is limited.

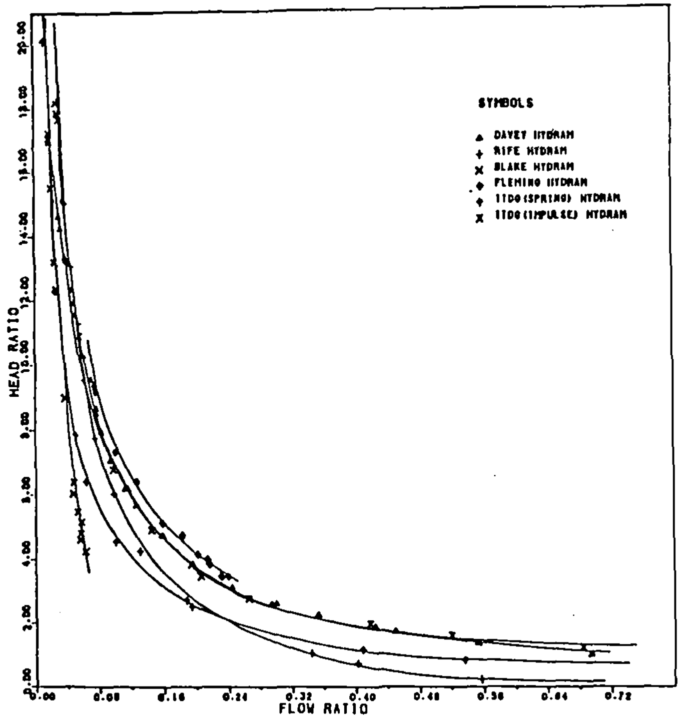

Fig. 4. Head Ratio vs. Flow Ratio for Various Hydrams, H=2m

8. Different hydrams of the same size, operating with maximum efficiency, have different pumping capacities which are proportional to the total supply flow requirements of the hydram. A well designed locally-made hydram has comparable operating characteristics as the commercial hydrams of the same general size.

9. The size of the air chamber has no significant effect on hydram operation particularly at small supply heads.

10. The size of the air valve is not critical in hydrams. A simple small-sized (< 1 mm) hole is sufficient.

COMPUTERIZED MODEL OF THE HYDRAM

In conjunction with the experimental tests, a simple, yet accurate, computerized model of hydram operation was developed. The theoretical derivation of this model is given elsewhere (Kahangire, 1984; Schiller and Kahangire, 1984b). The model is a more simplified version than that developed by Krol (1952) and is more precise than that of Iversen (1975) which contained few details on water hammer and friction head losses. A key feature of this model is that it must have inputed to it some parameters of the hydram which can be measured experimentally, or determined in the field, using steady-flow conditions. These parameters then contain information about the hydram that provide the link between the particular hydram and the generalized model. Tests were done with the model, and these were compared with actual operating characteristics measured experimentally on a locally-made hydram (Schiller and Kahangire, 1984b). The curves shown below indicate good agreement between model and experimental results.

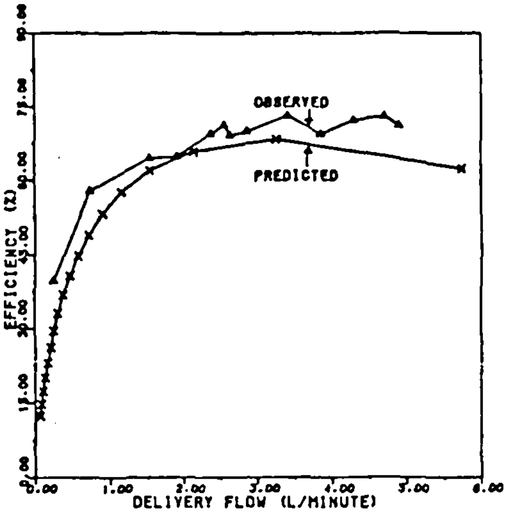

FIG.5 PUMP EFFICIENCY

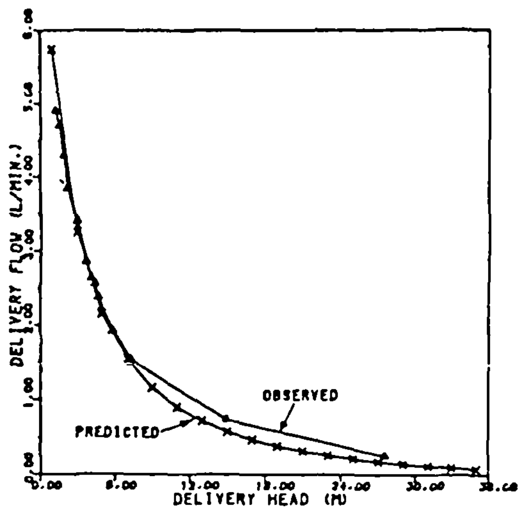

FIG.6 PUMP CAPACITY

The model can be used to assist in design and the proper site selection of hydrams. Figures 7 and 8 below indicate that influence of waste valve friction loss on pump capacity and the influence of supply head on pump efficiency. Curves such as these can assist the designer and can facilitate the choice of the best hydram for a given operating site.

HYDRAM CONFERENCE: ARUSHA, TANZANIA, JUNE 1984

Recently a hydram technology conference was held in Tanzania (June, 1984). The purpose of the conference was to promote the use of hydram technology in the developing world. The workshops discussed the operating characteristics of the hydram, the manufacture, operation and maintenance of hydrams, research methodologies and design needs in hydram development. Participants at the workshop visited Jandu Plumbers, a company in NorthernTanzania that is presently manufacturing hydrams. Field visits to operating hydrams were also made. An agenda of the workshop is appended. Further information about the workshop can be obtained from the International Development Research Centre, Ottawa, Canada.

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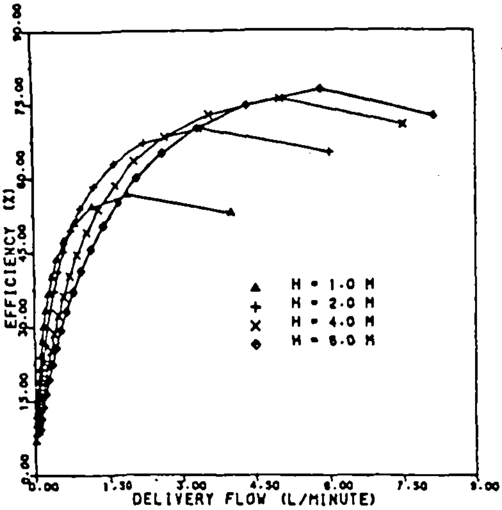

FIG. 7 EFFECT OF SUPPLY HEAD ON HYDRAM EFFICIENCY

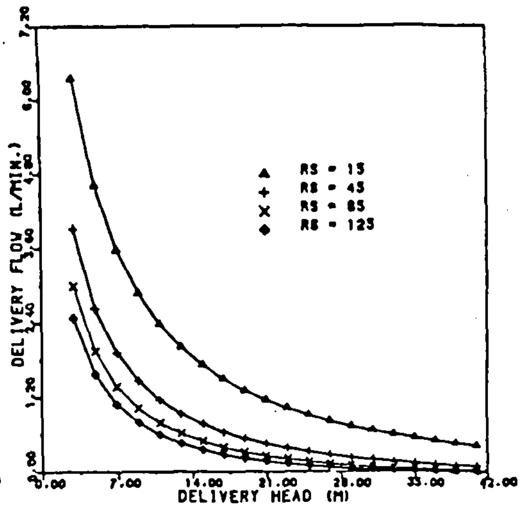

FIG. 8 EFFECT OF FRICTION HEAD LOSS OF THE WASTE VALVE ON PUMP CAPACITY

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Appendix I : HYDRAULIC RAM PUMP WORKSHOP

AGENDA

Tuesday, May 29, 1984

Mr. E.M. Ngaiza, Director General CAMARTEC, Arusha	Welcome Address
Mr. A. Redekopp IDRC Ottawa	General Introduction to IDRC and its program activities

Session I

The Hydraulic Ram Pump - Its History, Its Operating Characteristics and Its Potential Use

Session Chairman - Mr. David Tulapona, CAMARTEC

Dr. Eric Schiller, Professor, University of Ottawa, Canada

Session II

Reviews of Country Activities and Programs

Session Chairman - Mr. James Chauvin, IDRC

Country Reports

Wednesday, May 30, 1984

Session III

Manufacturing, Operations/Maintenance and Field Trip

Session Chairman - Mr. A. Baynit, CAMARTEC

Mr. S.S. Jandu Mr. M. Leach	The Manufacturer, Selection and Maintenance of Hydraulic Ram Pumps
Mr. S.S. Jandu	Visit to Jandu Plumbers Ltd. Manufacturing facilities
Mr. W.A. Baynit	Field trip to see hydrams installed in Arusha area

Thursday, May 31, 1984

Session IV

Research Methodologies and Research Needs

Session Chairman - Mr. A. Redekopp

Mr. Patrick Kahangire, Water Development Dept, Uganda

Mr. E. Th. Protzen, Institute of Production Innovation, Univ. of Dar es Salaam.

Installation, Operation and Maintenance

Mr. S.A. Mbwette, Faculty of Engineering, Univ. of Dar es Salaam

Mr. A. Mzee, Chief Maintenance Engineer, MAJI, Dar es Salaam

Dr. E.J. Schiller, University of Ottawa

Group Discussions	Ideas and Recommendations for Research and Development
Group Discussions continued	
Plenary - Summary presentations by group chairmen	Concluding Remarks

IWRA Vth WORLD CONGRESS ON WATER RESOURCES
9 - 15 JUNE 1985 BRUSSELS BELGIUM
WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 151 Aspect number 5

LONG-TERM WATER RESOURCES PLANNING FOR AGRICULTURAL DEVELOPMENT IN THE ISTRA PENINSULA, YUGOSLAVIA

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ABSTRACT

The Istra peninsula is situated on the extreme north-west part of Yugoslavia and it cover an area of 3100 sq. km. with about 200000 inhabitants. The western coastal belt and the Mirna and Raša valleys are extensive plains with terra rossa and alluvial soils of a thick top soil cover. This is suitable for high-yield crop development, particularly orchards and vegetables and have been the target of investigations and planning. Although the area benefits of relatively high rainfalls, because of unfavorable orographic and geologic conditions the major part of it is lost for the water resources, for which reason a lack of water for all needs have been felt for a long time. This is the reason that comprehensive investigation works comprising both underground water development and collecting of surface water by storage basin construction have been undertaken some years ago. The most serious limitations for intensive agricultural development is the deficit of the water in the vegetation season which ranges between 300 and 400 mm. yearly. Together with the proposed plan for long-term water supply for irrigation, a significant changement of the traditional cropping pattern have been considered. The hydrotechnical solution consists in construction of a douzine of storage basins which will satisfy the region with water up to the year 2015. It is estimated that by this time, owing to a very intensive touristic and agricultural development, particularly by introducing irrigation, a ninefold water consumption will occur. The most important estimates and computations are performed by using a set of mathematical models, foremostly the flow analyses, water requirements for irrigation, single reservoir operations, system reservoir operations, etc.

Keywords: Irrigation, storage basin, cropping pattern, water requirement, long-term planning, irrigation system, geologic conditions, pedological surveyings, mathematical models, simulation techniques.

INTRODUCTION

The Istra peninsula is situated in the extreme northwest part of the Yugoslav Adriatic coast with an area of about 3100 sq. km. It is the biggest peninsula in the Adriatic sea with most developed tourist trade in Yugoslavia.

The area benefits of a typical Mediterranean climate with cool and wet automns and winters and temperate and dry summers. While the central and eastern part of the peninsula have marked topographic features the western coast and the Mirna and Raša valleys are flat areas with a thick layer of terra rossa and alluvial soils, where intensive farming of high-yield crops can be developed, partcularly vegetables and orchards.

Yet in the years 1970, a set of surveyings, investigations and designings have been started as a bases for prepairing a long-term water resources developing plan /up to the year 2015./ to meet the needs for domestic supply and irrigated agriculture. In the period 1975-79., as UNDP/FAO Project, a first draft of the general hydrotechnical solution

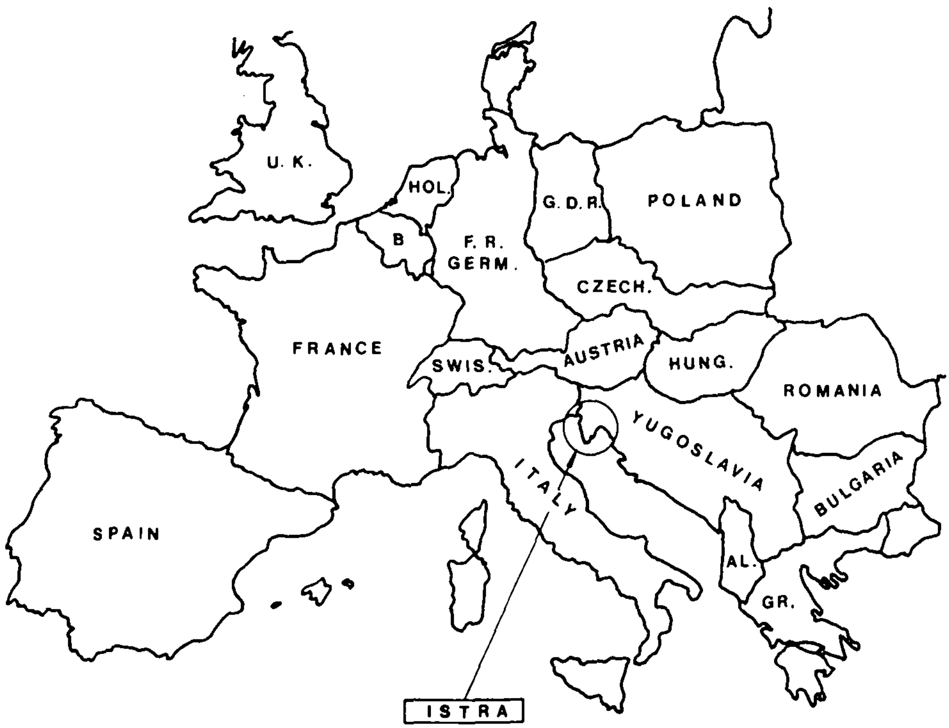

Fig.1. Location map

for the proposed water supply system have been elaborated. The study comprises the domestic water supply system for all inhabitants, tourism and industy, as well as irrigation of nearly 20000 hectars of land. The works have been continued after the year 1980 and are still in corse and include compilation of the final version of the study and detailed designs for the first stage of implementation of an area of about 2500 hectars. It is forseen that the works will be terminated at the end of 1984.

The most serious limitation to the development of intensive agriculture in the area is inadequate water distribution during the vegetation season. This problem is most acute along the southern coast, near Pula, decreasing toward north and north-east. Generally, the water deficit for the whole vegetation season and for most of major crops ranges between 300 and 400 mm. yearly on average.

THE PHISICAL ENVIRONMENT

As stated above, the area lies in the Mediterranean climatic region with average yearly temperature along the coastal belt of about 14 C. The rainfalls are dominantly of the orographic type, ranging from a minimum of about 700 mm. in the southwest to ~~about 2300 mm~~. on mountains peack at the northeast. These variations within a distance of mere 80 km. result from different orographic and climatic features, and particularly from an impact of wet air mases from the sea on the rear side of the Alpine range. Nevertheless, the region quite every year suffer of severe droughths durind the season's months of peack consumtion. From the other side, during the authomn and spring months, abundant precipitations are encountered provoking damages by floods in the river plains, and erosion problems in the hilly and mountainose areas. These uneven rainfall distributionn along the year imply planning of both irrigation and drainage systems, and in the river plains additional flood control works.

The region predominantly consists of two major geological formations: Cretaceous limestones and Terciary marls. The first one, mostly developed along the western and southern coastal belt, are characterized by typical karstic forms with considerable groundwater circulation and poor to negligent surface hydrography. The latter one, predominantly developed in the central part of the peninsula, are impermeable nearly through their extension and have a highly developed supface hydrography and extremely erodible cover.

As a result of different geological settings, two entirely different types of hydrological properties have been developed in the peninsula. In both cases, the available water resources are poor. In the former the rain water percolates fast to the impermeable bedrock and flows away toward the sea by usually unknown channels at depths which may vary with the season. In the latter, atmospheric water runs fast down the steep slopes in torrential streams carrying large amonts os suspended and bed load. Streams of this kind very often cause damages by flooding and silting up water engineering structures, roads and farm fields. In both cases some control mesures are necessary, first the investigations and studies and then engineering works to increase and improve the available water resources.

PROPOSED SOLUTION

Land and soil considerations

The basic data for long-term planning of agriculture in Istra were drown out from land and soil surveyings. These investigations reveale that the peninsula have a total of 22000 hectares of land class 1., suitable for farming without limitations with the only constrain of occasionally deficit of moisture in the vegetation period. For the selection of thr proposed irrigation fields, the following additional criteria have been taken into consideration:

i/ All soils of the terra rossa type which mostly consist the western part of the area are more suitable for irrigated agriculture than the other soils of the peninsula.

ii/ The planned crops /crops of high market value and forrage/ can be very successfully grown on the irrigated terra rossa.

iii/ The terra rossa soils are suitable for irrigated farming because:

- have good drainage properties, thus no salinity problems,
- are suitable for cultivation under modern technological conditions, and
- are located in flat area and don't require high investment for development.

iv/ To decrease the investment cost of the whole system the minimum irrigation unit of 200 hektares have been choosen. The land located at elevations over 150 m.a.s.l. have beenassign a lower priority order because of higher irrigation operating costs.

v/ The ratio between net and gross sutface have to be as high as possible - to decease development costs.

Water resources development

To prepare the feasibility study, a set of surveyings and preliminary reports were compiled with the view of defining the conditions for construction of storage reservoirs, use of ground water resources, water supply facilities for irrigation and domestic use, as well as irrigation systems. The investigations in agricultural development from the aspect of irrigated farming also included the establishment of three experimental irrigation fields.

The investigation results indicated the need for construction of a total of nine storage reservoirs to cover the needs until the year 2015. The plan provides for an increase in available resources by about ten times the amount available in 1975. This may greatly contribute to developing tourism and irrigated agriculture. The recomended water resources system anticipates the construction of a total volume of about 150 millions cubic meters of storage devided into nine units most of which located in the central flisch zone. Besides, all the

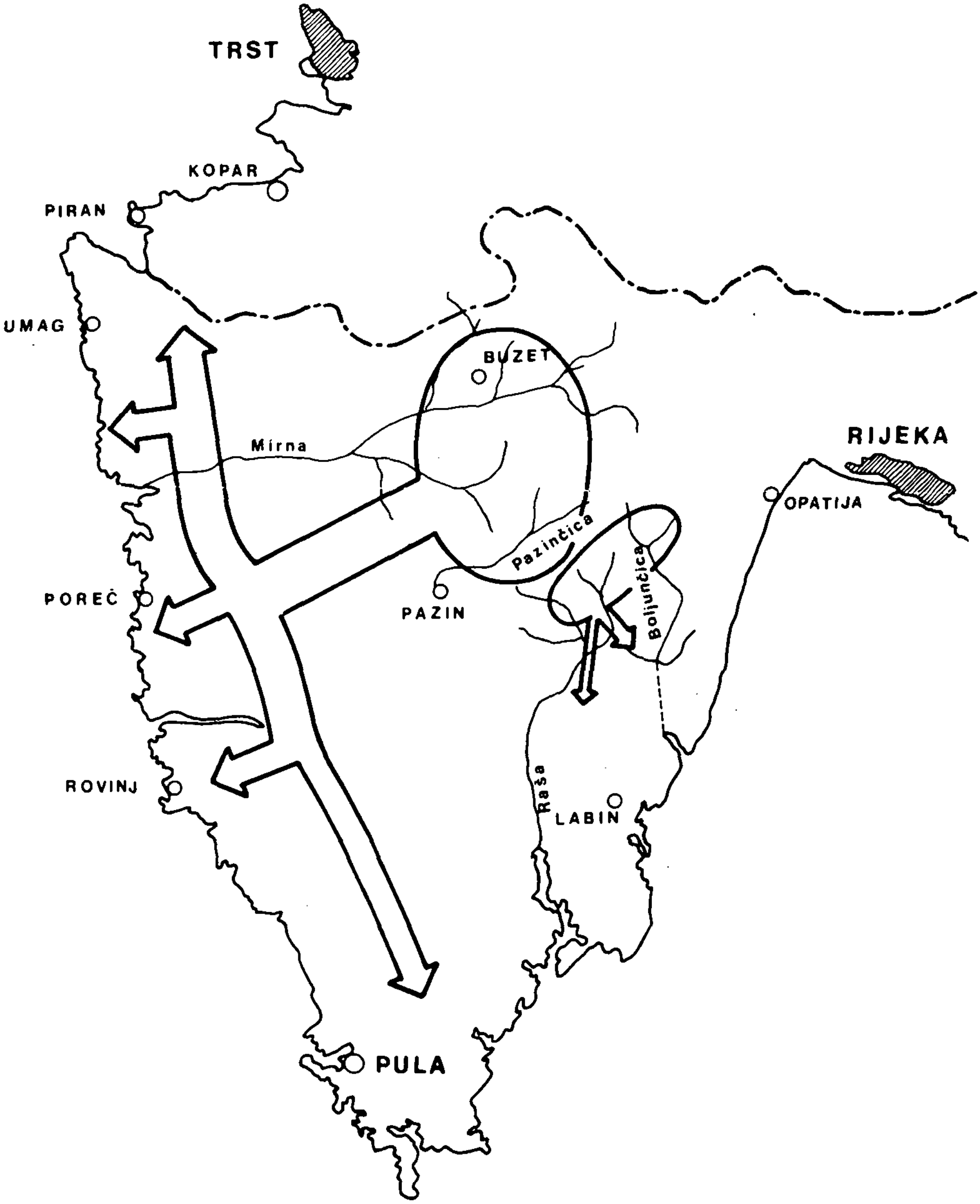

Fig.2. General lay-out of water resources development

available ground water would be tapped. The storage reservoirs and water supply systems are planned as multi-purpose structures for flood control, domestic and irrigation water supply. The selection of the best solution was performed on a base of set of alternatives with various sources and distribution nets. Simulating the operating cost of each alternative, the solution of least operation cost have been proposed for implementation.

As planning technology, a number of mathematical models in conjunction with computerized technique were used. These analyses were primarily used to determine the water consumption by crops, total montly and yearly requirements, single and multi reservoir operation, optimization of the distribution system and defining the sizes of stage units.

Staging the plan

The whole irrigation area is devided into seven distribution systems with 32 irrigation service areas, covering a total of 18960 hectares. Each of the single irrigation area make a technical and technological unit and mostly is considered for development in a single stage. The size of such a unit vary between 200 and 800 hectares, mostly 400-600 ha. For all of them, the cropping patterns, water requirements by each crop, capital and operating costs, irrigation efficiency and other estimates are defined. Based on cost-benefit estimate, the priority order is established for implementation of all the project systems and works.

The whole programme is forseen to be implemented into six stages, starting by 1985.

Table 1. Irrigation water requirements by stages in millions of cubic meters per year

Irrigation area	Yearly water requirements with 80% confidence by stage of development in mil. c.m./year					
	1 1985	2 1990	3 1995	4 2000	5 2005	6 2015
Bujština	5,46	5,46	5,46	15,89	19,07	19,07
Mirna valley	-	-	2,65	4,24	6,79	10,97
Poreština	-	-	-	5,37	18,33	21,93
Rovinjština	-	-	-	-	4,97	12.01
Mirna watershed total	5,46	5,46	8,11	25,50	49,16	63,98
Pulština	-	1,35	6,85	6,85	9,07	9,07
Raša - Čepić	-	-	2,60	7,80	7,80	9,83
Istra total	5,46	6,81	17,56	40,15	66,03	82,88

To define the economical parameters of irrigated agriculture, all cost component for growing each crop in each irrigation service area, both as dry farming and irrigated agriculture with different prices of water /from 2 to 6 Din./ have been computed. Simulating the running of the system for all combination of capital and operating costs, an anvelope curve of upper boundary of economical development for each crop have been defined.

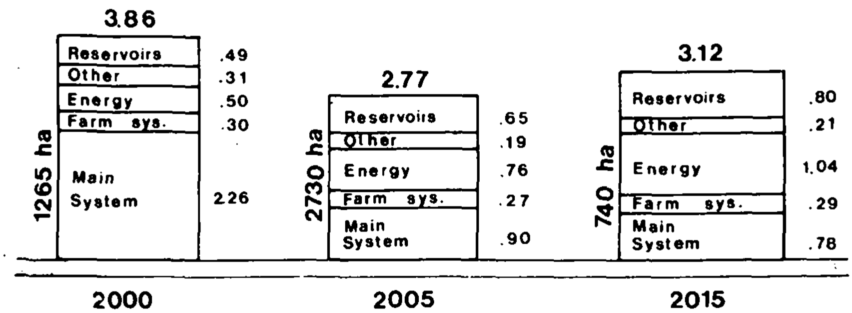

Fig.3. Average annual cost of water per cubic meter for Boreština by stage of development

In the second part of the plan, where sources of water were considered first of all the known and available water sources /springs/ have been used, The remainder of uncovered demand have to be supplyed from storage basins which will be constructed in adequate sections of the river valleys and torrents. A total of over 20 potencial solutions were considered, out of them after simulating of single and multi reservoir operations, a set of nine of them have been proposed for implementation.

CONCLUSIONS

Presently, the only significant water user in Istra is the domestic and industrial water supply system, while irrigation will become an important user only after 1995. Todays water requirements are covered from karstic springs in the river valleys and from dug wells around Pula. Most of the future water requirements will be met from surface runoff and storages of spring water formed during high flows to meet requirements of domestic, industrial and agricultural users. This is the main reason why all future water resources development projects should take into consideration the combined requirements of domestic and agricultural users and try to meet their needs jointly.

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IWRA Vth WORLD CONGRESS ON WATER RESOURCES
9 - 15 JUNE 1985 BRUSSELS BELGIUM
WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 153 Aspect number 8

AN OPERATIONAL MODEL FOR ASSESSING WATER SUPPLY: CONSTRAINTS AND AGRICULTURAL-ENERGY CONFLICTS

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ABSTRACT

A developing concern in parts of Canada is the impact of new water using developments on both existing areas as well as on the potential for other developments, including agriculture, energy and industrial, in the future. A model has been developed for Environment Canada to examine the constraints on developments arising from limited water availability. The model compares monthly water availability and demands at a number of locations throughout a basin or basins and produces statistics of failure.

Water demands are related to level of economic activity in each major sector of the economy and the intersectoral effects can be handled using an input-output matrix. Changes in activity in each region arising from demands in the other regions of Canada are dealt with in a similar way. As well as the demands arising from the growth in each major economic sector the model will also handle specified individual large-scale users, such as major energy-related projects. At present the demands are deterministic based on the level of economic activity, actual water demands where available, population levels for municipal supply, and area and crop type for irrigated land. The model runs on an IBM-PC and has a modular structure so that each major component can be upgraded as required.

Keywords: Modelling, water demands, input-output, energy, agriculture, municipal, industrial, irrigation, personal computer.

INTRODUCTION

The federal government in cooperation with the provincial governments has a responsibility to maintain a reconnaissance on the water availability in the different regions in the country vis-a-vis the current and potential future requirements for the water. Also, in negotiating international water agreements it must ensure that the long term water requirements of Canadians are met.

A current thrust of federal policy is to ensure the production of fossil fuels within Canada. A number of sources being considered have a high water demand for separation and processing. Because of the capital intensive nature of many of these projects, they will have a very high priority during water short periods.

Concurrent with expansion of major energy-related projects other demands are expected to grow and growth in irrigation in particular has the potential of competing for the same sources as energy development.

Following a compilation of information on major proposed energy-related uses (Reference 1), Energy, Mines and Resources sponsored the development, by the Inland Waters Directorate of Environment Canada, of a forecasting tool which could be used in a planning capacity to evaluate the effect of specific major development as well as alternate growth patterns on water shortages. The model is designed to make maximum use of data already obtained by agencies such as Water Survey of Canada and Statistics Canada (Statscan).

The model is being developed in phases and consists of a central core to which various modules can be added to upgrade water demand forecasting in the various sectors. Water demands based on population forecasts and industrial and agricultural growth are compared to water availability at various nodes and output data on shortfalls generated.

In this paper the concepts and structure of the model are summarized and typical results presented for the model as it presently exists. These resultswere obtained using the South Saskatchewan Basin as a test case. A more comprehensive description of the model is contained in a report to Environment Canada (Reference 2).

CONCEPT OF THE WATER MANAGEMENT MODEL

The basic purpose of the model is to compare forecasted water use requirements with available supply to identify those basins and subbasins which have potential water shortages. The area to be analyzed is divided into subbasins, typically the basin of a tributary or segment of a major river system. These subbasins can be interconnected or completely separate depending on the area of interest. At the downstream end of each subbasin data on flow are required, so that typically a gauge location is selected. These "gauge points" are used to describe the pattern of available surface water. The results of the subbasin analysis are aggregated to produce results by major river basin and by province or economic region. The economic regions used in the model are the five regions used by Statscan: Atlantic, Quebec, Ontario, the Prairies and British Columbia.

The model may be considered as a 3-step process illustrated in the flowchart of Figure 1.

Step 1

Determine the water use requirements of each subbasin. Water use data include both gross intake and consumptive use. Gross intake includes all water abstracted from the stream and groundwater system within the subbasin whether or not the water is returned to the subbasin. Consumptive use is that water abstracted and not returned to the subbasin. Data files are prepared containing base year information relating to population, agricultural and industrial development, as well as the water use rates for each of these categories. Forecast files are also prepared by the user describing the growth anticipated in each category. Minimum flows can be specified at the outlets to each subbasin to reflect water quality and recreational water use requirements within the subbasin.

Step 2

Determine water availability. For each subbasin, a sequence of naturalized monthly hydrologic records is prepared.

Step 3

Compare water use and availability. For the demand forecast year selected by the user, the model determines the water intake and consumption requirements and compares these month by month against the naturalized hydrologic record of the subbasin. Statistics relating the gross intake and consumption to water availability are produced.

These three steps are described in more detail below.

Water Demand Requirements

Water demand is determined for five basic user categories: municipal, agricultural, industrial, minimum flow constraints and special developments. The data required by the model to determine the water use by each of these five categories are discussed below.

Municipal

Municipal water use in the model is expressed as litres per capita per day for both rural and urban populations. Within each of these two categories it is possible to specify the percentage of per capita demand contributed by residential, commercial and public sectors.

The per capita water use may be varied from month to month, for example to simulate a higher monthly requirement during the summer period. Provincial municipal water use data are available from the Canada Water Year Books produced by Environment Canada (Reference 3). For specific subbasins it may be possible to obtain more accurate data from other published sources or from the largest municipality in the subbasin. It is important that industrial water requirements, supplied through the municipal distribution system, be eliminated from the reported municipal requirements. Industrial water use is accounted for separately in the model.

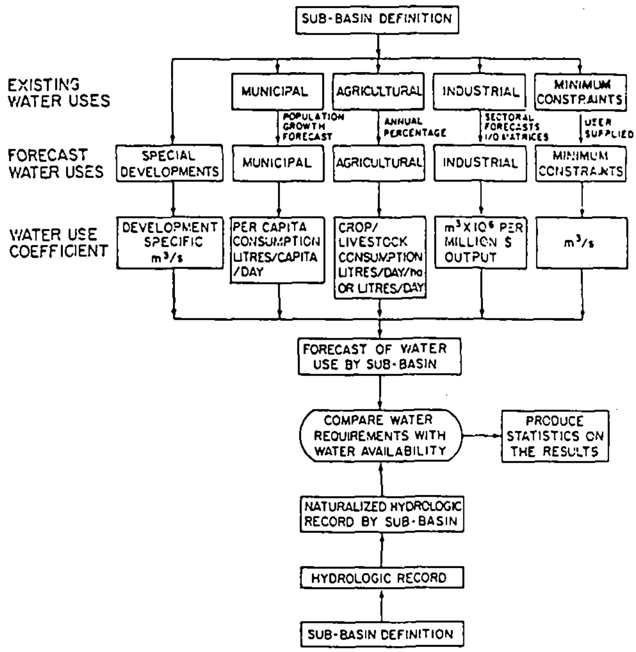


FIG. 1
DATA FLOW DIAGRAM OF MODEL

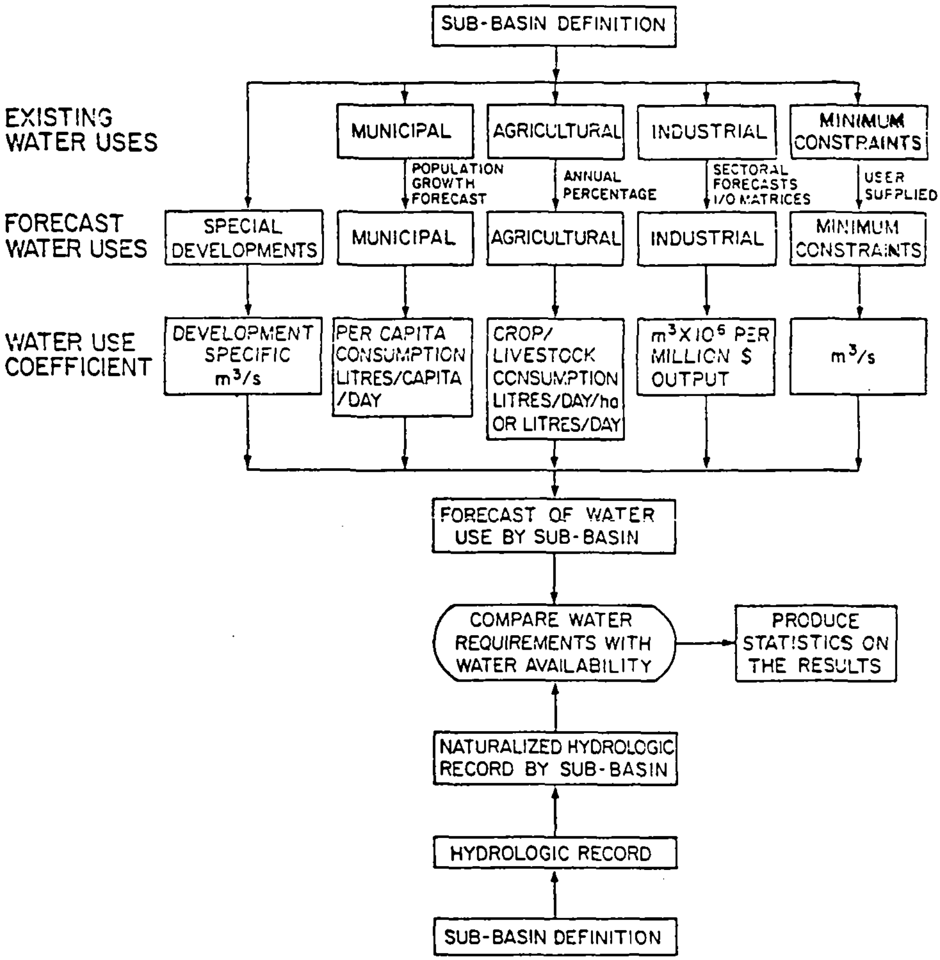


FIG. 1
DATA FLOW DIAGRAM OF MODEL

Population data for the model, split into rural and urban components, are available from Statscan based on the 1981 census (Reference 4). Statscan can also provide forecast population growths by province under various development scenarios which are apportioned to the subbasin on the basis of baseline population data.

The model uses the municipal water use rates and the forecasts of future population to establish the total municipal requirement in each subbasin.

Agricultural

Water use for the agricultural component is expressed as gross intake and consumption in cubic metres per hectare per year for a variety of irrigated crops. These demands are distributed over the year using monthly distribution factors corresponding to the cropping calendar. Livestock total intake water requirements are expressed in litres per head per day and consumption is expressed as percentage of the total intake. Water use coefficients per head or per hectare are specified by the user. The coefficients are highly variable from area to area and from year to year due to differences in precipitation, evapotranspiration, irrigation method and, of course, crop type. Plans are currently underway to develop an agricultural submodel compatible with the main model which will compute a monthly time series of irrigation requirements over the full length of hydrologic record by subbasin and by crop on the basis of meteorologic and crop characteristics.

Input data include the base year total irrigated area by crop and number of head by livestock type for each subbasin. Data are available for the 1981 census from Statscan (Reference 5). However, only total irrigated area for all crops is reported. Other sources must, therefore, be used to determine the irrigated area was not warranted at this stage, due to a general lack of data.

It is also possible to simulate the transfer of water from one subbasin to another or from outside the major basin under study. This enables the user to study the effects of a diversion on satisfying needs in water short areas.

Comparison of Water Supply and Water Use

The computational module of the computer program is very straightforward. The model examines each subbasin in turn working from upstream to downstream in a hierarchical relationship defined by the user. For each month, the model performs a water balance analysis within each subbasin. Two ratios are computed as follows:

- intake/supply where intake is the gross volume of water withdrawn from the surface and subsurface system in the month

- consumption/supply where consumption is the volume of water consumed by the process or made unusable for reuse downstream.

The frequency of occurrence of these ratios in specified ranges is computed as the model examines each month in the hydrologic sequence. Note that the frequency of violations of the specified minimum flow constraint in the subbasin is also documented.

The model examines the upstream subbasins first, passing surplus water to the next subbasin downstream. Local inflows to the subbasin under consideration, diversions and surplus flows from upstream are considered in computing water availability. In this way, the impacts of all upstream water sources and water uses are accounted for in analyzing downstream basins. Final results may be aggregated by subbasin, basin, province or economic region.

Applications of the Model

The model has been designed to enable the user to ask a wide range of "what if" questions relating multisectoral growth to the availability of water. The model can also be used to make a preliminary assessment of various remedial measures (such as diversions) envisaged to relieve areas with chronic water shortages. The following specific applications have been identified:

(a) Primary application of the model is expected to be in the evaluation of water resource impacts of adding new energy-related developments to specific riverbasins. The model has been designed for this type of site-specific evaluation, and guidelines for use where specific data are not available to the water use rates for the various forms of energy developments are presented in the summary report (Reference 2).

(b) By changing the development forecasts, the impact of various growth scenarios on potential water shortage areas can be examined. The exogenous forecasts for each industrial sector are input by economic region. Therefore, the impact of industrial growth in one region can be studied in light of its effect within the same region and, through the interregional input-output matrices, on the other economic regions in the country. Similarly, the effects of change in one industrial sector or other industrial sectors in the region can be studied using the intra-regional input-output matrices.

(c) Population and agricultural forecasts can be altered to examine their impact on overall water availability and, hence, on water available to industry.

(d) The range of coefficients relating industrial water use to value of outputs in dollars has been developed from an examination of past trends. If the user wishes to examine the impact of an envisaged technological change, the range of the appropriate water use coefficients can be altered.

(e) Interbasin transfers can be simulated with the model and therefore the impact of planned diversion schemes on water shortage problems can be judged.

(f) The impact of additional on-stream storage on water shortages can be judged. Although the model does not currently simulate reservoir operation, the user may examine the sequence of water deficits and surpluses at a given location using mass balancing techniques. The impact of additional storage in alleviating the deficits can then be determined.

(g) Studies have and will continue to be conducted on medium- and long-term climatic changes in Canada and around the world. With the implementation of an agricultural submodel, it will be possible to evaluate the impact of forecasted climatological changes on crop water requirements and hence on overall water resource utilization.

TEST CASE

General

The test case discussed in this section is intended as an introduction to the application and capabilities of the water management model. Variations in water use coefficients and future development scenarios were investigated for the test basin. The chosen scenarios are intended to illustrate the flexibility of the model for analyzing the impacts of a wide range of options. The various scenarios which were analyzed demonstrate the potential of the model for identifying possible future water use problems. The sensitivity of the resulting water use problems to variations in future development and water use rates have also been analyzed.

The South Saskatchewan River Basin was one of the basins selected to test the model. The basin outline and the subdivision of the basin used in the test runs are indicated in Figure 2. This basin already had very substantial demand on it and a number of future energy developments are proposed.

The nature of this paper does not permit a full documentation of the test case which is available in Reference 2, however, some key results will be discussed. The scope of the test is indicated in the run strategy diagram of Figure 3. The variables examined are listed on the left side of the figure and the figure itself shows the values of each variable selected for each scenario. Generally, when examining the impact of changes in one variable, all other variables were held constant.

Questions Addressed

The most significant issues addressed in the test case were as follows.

(a) The effect of each of 3 agricultural growth scenarios - low (expected), medium and high.

(b) The effect of 3 specific development scenarios. The specific developments which were included are those energy-related water uses which were identified and documented in the Phase I segment of the current study.

(c) The two possible types of water transfer were studied.

- Transfers from node to node within a specific network. These transfers will not significantly affect supplies downstream from the junction of the two subbasins.

- Transfers from an external node to a node within the network. These transfers will affect the total supply within the total basin.

For the test case, a strategy of combining various options was developed as summarized in Figure 3. As noted, these items that were varied included:

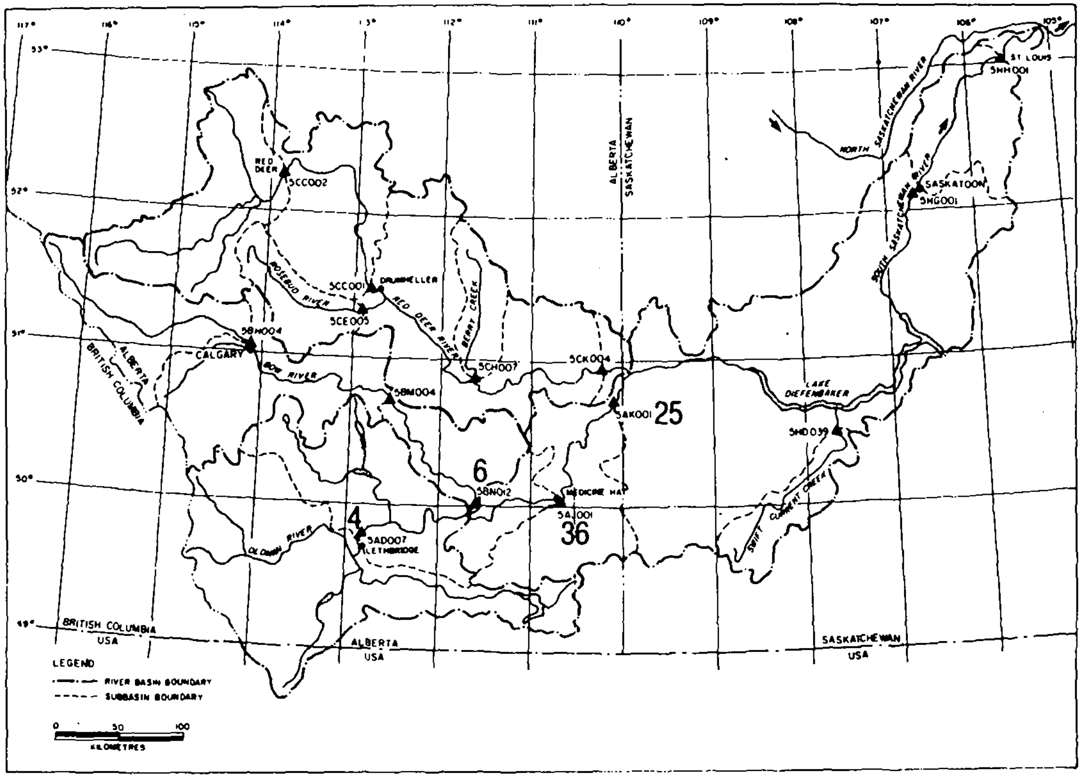
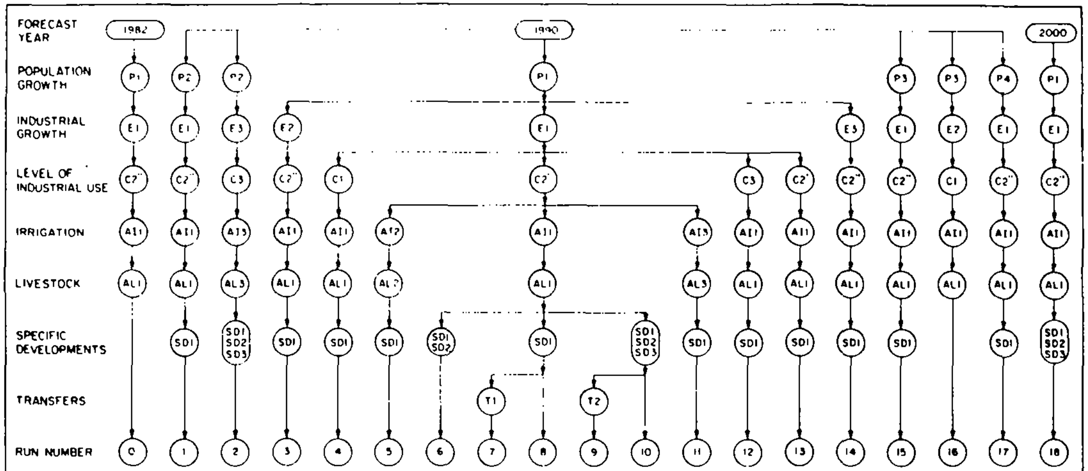


FIG. 2 NODAL POINT LOCATIONS AND DRAINAGE BASINS FOR THE SOUTH SASKATCHEWAN RIVER BASIN



LEGEND

P1 FERTILITY INCREASE AND SOME MIGRATION
P2 FERTILITY INCREASE AND MORE MIGRATION
P3 FERTILITY DECREASE AND SOME MIGRATION
P4 FERTILITY DECREASE AND MORE MIGRATION

E1 ECONOMIC COUNCIL OF CANADA
E2 U OF T NATIONAL FORECAST
E3 U OF T WESTERN FORECAST

C1 LOW USE DEFAULT COEFFICIENTS
C2' EXPECTED USE DEFAULT COEFFICIENTS
C2'' CALIBRATED USE COEFFICIENTS
C3 HIGH USE DEFAULT COEFFICIENTS

AI1 GROWTH IN IRRIGATION - LOW (EXPECTED)
AI2 GROWTH IN IRRIGATION - MEDIUM
AI3 GROWTH IN IRRIGATION - HIGH

AL1 GROWTH IN LIVESTOCK POPULATION - LOW
AL2 GROWTH IN LIVESTOCK POPULATION - MEDIUM
AL3 GROWTH IN LIVESTOCK POPULATION - HIGH

SD1 SPECIFIC DEVELOPMENTS - PLANNED 1980 - 1988
SD2 SPECIFIC DEVELOPMENTS - PLANNED 1989 - 1993
SD3 SPECIFIC DEVELOPMENTS - PLANNED 1994 - 2000

T1 TRANSFER NODE TO NODE WITHIN STUDY BASIN
T2 TRANSFER EXTERNAL NODE TO INTERNAL NODE

QUESTION ADDRESSED	RELEVANT RUN NUMBERS
POPULATION GROWTH	1,8,15,17
INDUSTRIAL GROWTH	3,8,14
LEVEL OF INDUSTRIAL USE	4,8,12,13
AGRICULTURAL GROWTH	5,8,11
SPECIFIC DEVELOPMENTS	6,8,10
TRANSFERS	7,8 AND 9,10
DEVELOPMENT WITH TIME	0,8,18
"GREATEST" AND "LEAST" USE	2,16

FIG. 3 ALTERNATIVE TEST CASE SCENARIOS FOR THE SOUTH SASKATCHEWAN RIVER BASIN

- population growth
- industrial growth
- water use unit industrial output
- irrigation growth
- livestock growth
- number of supplied major new developments.

Runs were carried out using 55 years of hydrology and for forecast demands for 1982, 1990 and 2000. For the industrial water use as well as "default" coefficients derived from published data, water use coefficients calibrated using actual basin data were tested.

For the South Saskatchewan Basin those variations having a substantial impact on the deficiencies were the specified major developments and agricultural development, and the combination of rapid growth of both lead to numerous "deficiencies" at a number of gauge points.

CONCLUSIONS

In summary, the model is capable of comparing various scenarios of development over an extensive geographic area giving a measure of potential future conflicts arising from normal growth on specific developments. Because of the importance of agricultural demands and the negative correlation between these demands and the surface flows, additional refinement of this aspect of the model will significantly improve its accuracy.

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ENVIRONNEMENT, DEVELOPPEMENT ET ETUDES D'IMPACT APPLIQUES AUX AMENAGEMENTS HYDRAULIQUES EN PARTICULIER DANS LES PAYS EN DEVELOPPEMENT

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RESUME

Entreprendre de grands travaux hydrauliques, en particulier dans les pays tropicaux en développement, entraîne fréquemment des perturbations étendues dans la région dans laquelle ils sont établis, par leur présence même, leur construction puis leur fonctionnement.

Des études d'impact sont alors nécessaires pour déterminer dans quelle mesure il est possible de minimiser les conséquences négatives du projet sur le milieu naturel et humain, ou, au contraire, d'en développer les potentialités favorables. Une surveillance attentive des répercussions du projet aux différentes étapes de son établissement doit permettre de connaître l'état de l'environnement à un moment donné et d'essayer d'en prévoir l'évolution, en prenant alors les mesures pratiques adéquates, secteur par secteur tout en assurant les coordinations nécessaires.

Cela peut se faire par l'adoption d'une législation nationale sur l'environnement, sous réserve qu'elle soit accompagnée des moyens pratiques qui en permettent l'application. Mais le caractère multiple des interventions à envisager et leur variété exigent souvent des décisions immédiates que, seul, un organisme ayant l'autorité, la compétence technique et les ressources financières peut prendre, tout en répondant aux aspirations des populations locales directement concernées. Les organismes de coopération et de financement doivent en favoriser la création, dans le cadre des structures politiques et administratives nationales, et ceci dès l'origine du projet.

Mots clés : Environnement-développement-aménagements hydrauliques-pays tropicaux-études d'impact-matrices-milieux naturels-milieux humains-hydrologie-géophysique-pédologie-agriculture-reboisements-pisciculture-collectivités rurales-transferts de village-reclassement-compensations-autorité locale-législation-coordination.

INTRODUCTION

Les grands travaux hydrauliques se sont multipliés dans le monde pendant les deux dernières décades, en particulier dans les pays en développement, sous l'effet de besoins croissants en terres cultivables et en énergie. Nombre d'entre eux ont entraîné des perturbations considérables dans le milieu naturel et humain - l'environnement - des régions dans lesquelles ils ont été établis.

Cette situation est maintenant évidente aux resposables des gouvernements concernés, d'autre part aux maîtres d'ouvrage et aux organismes de coopération et de financement. Des efforts sont faits pour : 1° analyser ces perturbations d'une façon systématique;
2° les corriger, voire les transformer en éléments de développement technique, économique et social. La présente note a pour objet de clarifier certains aspects de ce problème en insistant sur un point important : alors que les méthodes d'analyse de l'environnement se sont perfectionnées, le moyen de corriger les dommages causés au milieu sont encore trop souvent considérés comme secondaires car le responsable du projet a pour priorité la création et la mise en oeuvre de celui-ci. C'est sur ce point précis qu'un échange d'idées et d'expériences pourrait avoir lieu, au bénéfice des représentants des gouvernements concernés et des concepteurs de projets.

La présente note ne traite pas de l'impact du projet lui-même sur l'économie nationale, ce qui est un problème fondamental, mais nécessite une autre approche. Elle suppose que les décisions concernant : le principe de la création de l'aménagement, sa structure générale et son emplacement ont été déjà définis et les choix opérés entre les différentes alternatives possibles; seules des modifications mineures peuvent leur être apportées.

Un grand aménagement hydraulique bouleverse la vie d'un pays ou d'une région

En effet, les objectifs de cet aménagement sont d'améliorer la situation globale - économique, sociale, humaine, voire même les structures de la terre et des eaux - du pays ou de la région. Voici quelques exemples d'aménagements hydrauliques :
- Travaux de correction et de régulation de fleuves, de rivières, augmentation du régime de certains fleuves (canal de Jonglei au Soudan);
- barrages à objectifs uniques ou multiples : production d'électricité, irrigation, écrêtement de crues et régularisation de cours, compensations hydrauliques (lâchures);
- barrages pour création d'usines marémotrices (La Rance, France), périmètres d'irrigation, draînages de terre inondables et de marais;
- digues à objectifs divers, par exemple pour protéger une région et assécher des terres (Pays-Bas), pour empêcher les remontées de sel (Barrage de Diama, Sénégal);
- lacs collinaires;
- forages hydrauliques, pour recherche d'eau potable, ou création d'abreuvoirs;
- réservoirs d'eau potable, adductions d'eau.

La réalisation de l'aménagement va changer l'environnement de la région :

* en introduisant dans un milieu rural parfois fragile des entreprises qui vont par leur présence, leurs travaux et l'emploi de travailleurs

étrangers au pouvoir d'achat élevé, bouleverser les structures sociales traditionnelles;
* en substituant à l'écosystème primitif (avant l'aménagement) un autre écosystème, ainsi :
- à la place d'une vallée il y aura un lac de retenue qui va altérer les caractéristiques locales climatiques (création d'un microclimat, du par exemple à l'accroissement de l'évaporation)géophysiques (morphologie de la région, volcanisme, glissements de terrain) pédologiques (structures des sols, sédimentation, dépôts d'alluvions, stérilisation par submersion de sols fertiles, de sols forestiers) entraîner des modifications de la flore et de la faune ou la destruction de sites archéologiques ...

A la place d'un marais, il pourra y avoir des terres cultivées, des eaux courantes à la place d'eaux stagnantes, un assèchement des terres consécutif à l'établissement d'un système irrigation/drainage, une modification chimique de la composition chimique des sols (remontées de sel ...). Un problème très important (Nil, Sénégal) : celui des limons et alluvions qui couvraient les terres au moment des crues et qui seront retenues désormais par le barrage.
* en modifiant l'écologie humaine de la région (démographie, économie, structures sociales, ainsi :
- la création d'un lacde retenue va faire partir les gens dont les villages seront inondés (Maung); des agriculteurs vont perdre leurs terres fertiles (rizières de bas-fonds inondés), des éleveurs leurs terrains de parcours. La structure sociale va être détruite; le réseau de communications va être bouleversé par la création de routes carrossables,l'afflux de travailleurs va transformer l'économie de subsistance de la région en une économie de marché, précipitant ainsi l'exode rural et la disparition de civilisations locales.

Un aspect important est l'expansion possible de maladies tropicales par rupture des équilibres écologiques ancestraux et favorisant le développement de vecteurs parasitaires (moustiques, escargots, mouches diverses, etc ...).

Mais ces bouleversements peuvent hâter des changements nécessaires en accélérant la modernisation de la région, en créant de nouvelles possibilités d'emploi, en soulageant la pression démographique, en favorisant des investissements, en "désenclavant" des sites isolés montagneux, l'aménagement constituant un "point focal de développement".

Néanmoins les avantages attendus de l'aménagement profitent rarement à la population locale.

Ainsi la production électrique d'un barrage va rejoinddre le système général de distribution d'électricité, les travaux d'irrigation enrichissent des terres parfois lointaines, la régulation d'un fleuve peut n'apporter aucune amélioration de la condition des riverains, parfois au contraire la déteriorer : suppressions des cultures de décrues dans la vallée du fleuve Sénégal, de l'économie des marais dans le "sud" soudanais, multiplication des forages abaissant la nappe phréatique etc ...

Tous ces problèmes sont liés entre eux, mais on peut supposer que sur le plan national la balance entre avantages et inconvénients d'un aménagement est positive puisque la décision a été prise de l'entreprendre. Le rôle des études d'impact est alors désicif car elles permettent de se rendre compte de la situation prospective de la région, pendant l'établissement de l'aménagement, puis lorsqu'il sera en fonctionnement.

Environnement : les principaux problemes sectoriels

Ainsi les problèmes qui peuvent accompagner la mise en oeuvre d'un grand aménagement hydraulique sont innombrables et complexes. De plus ils se recoupent, parfois se chevauchent et évoluent avec la succession des opérations. Leur seule analyse pose à l'écologiste une tâche difficile.

Différentes méthodes sont utilisées dans ce but, la plus simple étant de faire ressortir à travers une ou plusieurs "matrices" les conséquences des actions successives qui commandent la réalisation du projet sur la situation du milieu naturel et humain.

Les principaux problèmes sectoriels qui se posent à l'écologiste étudiant, par exemple, les conséquences sur l'environnement de la création d'un grand barrage, ont été maintes fois définies. Des listes de questions ont été préparées, des exemples de matrices mis au point. Voici comment celle dressée par la Commission Internationale des Grands Barrages définit les problèmes d'environnement qui se posent à l'occasion d'un barrage :

En premier lieu elle énonce : les objectifs de l'aménagement (irrigation, production d'électricité, régulation des crues, navigation, etc...), les types d'actions à entreprendre : construction du barrage, dérivation des eaux, déboisement, ouverture de carrières; les zones de marnage, zones en aval et en amont du barrage, etc ...

Ensuite elle énumère les impacts prévisibles : sur le climat et le cadre géophysique (morphologie, sols, stabilité des pentes, assèchement des terres ...) ; sur la situation hydrographique : débits finaux, évapotranspiration et évaporation, qualité des eaux, situation de la nappe phréatique; sur la flore et la faune terrestre et aquatique; sur la situation démographique, économique et sociale : exode rural, emploi, évacuation des villages, compensations, transformation de l'agriculture et des forêts, développement de la pêche et du tourisme, du commerce et de l'industrie, création et suppression d'emplois; sur les structures fonçières et collectivités rurales.

Enfin la matrice définit un certain nombre d'actions à prendre comme conséquence de l'analyse ci-dessus : compensation et réinstallations des personnes déplacées, aménagements agricoles, forestiers, piscicoles, contre-barrages, infrastructure touristiques, création de petites industries, lutte antiérosive, traitement des eaux. Ce sont toutes des actions sectorielles qui constituent la conclusion de l'étude d'impact.

Mais cette matrice a un caractère général qui ne donne pas une vue prospective et dynamique de l'ensemble de l'évolution de l'environnement de la région. En fait plusieurs matrices doivent être conçues, correspondant chacune à une étape importante de la réalisation du projet.

Les etudes d'impact

Une étude d'impact a pour objet "d'étudier de manière approfondie les conséquences d'un projet sur le paysage, les milieux naturels, l'air, le sol et l'eau, la faune, la flore, ainsi que sur les populations concernées". Elle doit comporter : une analyse de l'état initial du site, des effets du projet sur l'environnement, des raisons pour lesquelles, de ce point de vue, le projet a été retenu, enfin des mesures qui doivent être prises pour compenser les conséquences dommageables du projet sur l'environnement, y compris l'évaluation du financement nécessaire. (Définition du décret (français) du 12 octobre 1977.)

Ainsi l'étude d'impact doit répondre aux problèmes posés par les conséquences du développement sur l'environnement. Elle a un caractère non seulement descriptif, mais également dynamique car elle doit proposer des solutions aux problèmes qui se posent et en évaluer le coût. Il appartiendra alors à un organisme compétent de prendre les mesures nécessaires correspondantes.

Mais la définition donnée ci-dessus de l'étude de l'impact d'un projet n'est pas suffisante, car elle ne tient pas compte des conséquences du projet sur l'environnement local pendant son établissement et sa mise en oeuvre. En fait, plusieurs études d'impact doivent être entreprises dans le cadre d'un grand aménagement hydraulique.

Une matrice appropriée constitue un guide pour une étude d'impact, mais ne saurait la remplacer; l'utilisation d'un système matriciel constitue sans doute le moyen le plus convenable pour aborder le caractère global de l'étude d'impact sans en négliger les éléments importants. Mais chaque stade du projet implique des changements qui doivent se refléter dans la matrice. Cela demande la préparation d'une matrice particulière correspondant à la situation du projet à un moment précis de son évolution et qui va aboutir naturellement à une série d'études sectorielles dont les résultats devront faire l'objet de traitements séparés.

Etude d'impact et évolution d'un projet

Lors de l'établissement d'un grand aménagement hydraulique qui peut s'étendre sur une dizaine d'années, des études préliminaires au fonctionnement satifaisant et équilibré, certains problèmes surgissent, puis disparaissent au cours de travaux. C'est le cas, par exemple :
- des perturbations engendrées par l'afflux d'une population ouvrière qui peut atteindre un millier de personnes, mais qui doivent s'atténuer, puis disparaître une fois la construction terminée;
- des problèmes posés par l'évacuation des villages condamnés à la submersion lors du remplissage du lac de retenue, et qui, même s'ils ont été soigneusement examinés, vont atteindre leur intensité maximum lorsque l'eau commencera à noyer les terres et les villages. Il faut espérer alors que la réinstallation des gens aura été prévue, et les compensations de leurs pertes matérielles déjà payées. Le problème peut se poser ailleurs, mais plus dans le périmètre de l'aménagement (cas d'un programme de "transmigration" de Java à Sumatra en Indonésie);
- de l'aménagement des abords de la retenue, des zones de marnage et su bassin versant non submergé, problèmes qui trouveront des solutions d'autant plus fructueuses qu'elles auront été préparées à l'avance;
- des aménagements temporaires nécessaires au chantier principal : routes d'accès, carrières, fabrique de ciment, cités d'hébergement, parcs d'engins lourds : ces aménagements peuvent ne pas disparaître avec la fin du chantier, mais au contraire se révéler utiles au développement de la région dont la modernisation va recevoir

un "coup de fouet" en démolissant les structures traditionnelles - physiques, économiques et sociales - et en en établissant d'autres.

Ainsi pour un même projet, plusieurs études d'impact seront nécessaires (ou plusieurs phases dans l'étude générale de l'impact du projet) :

La première fait partie de l'étude générale de faisabilité du projet : elle doit examiner, en plus de l'étude des critères généraux qui gouvernent le projet, son emplacement, les différentes solutions alternatives, les obstacles à prévoir.

La seconde étudie le dessin et le détail de la réalisation du projet, la recherche des meilleurs paramètres pour en atténuer les effets négatifs sur l'environnement, les solutions alternatives pour les annexes (routes, usines, carrière, etc ...) dont chacune fera l'objet d'une étude particulière.

La troisième est principalement prospective : elle examinera l'aménagement comme nouvel écosystème se substituant au précédent (par exemple le lac de retenue se substituant à la vallée originale). Elle comparera par suite chaque élément de la situation primitive de la région avec ce qu'on peut imaginer de la nouvelle, une fois le projet en fonctionnement.

Dans la pratique ces trois études d'impact ne se dérouleront sans doute pas dans un ordre chronologique précis. Elles donneront naissance à toute une série d'études techniques partielles : sédimentation, salinité, reboisement, lutte contre les parasites, etc ... qui peuvent évoluer également, avec l'état de l'aménagement et qui, à cnaque grande étape, seront coordonnées avec les autres études.

Dans les pays en développement : priorité au développement ou à l'environnement ?

Comment la situation apparaît-elle dans les pays industrialisés et dans ceux qui sont en développement rapide sur le plan de la protection de l'environnement ?

Dans les pays industrialisés et qui disposent de ressources financières et techniques importantes, un projet de grande envergure ne peut être entrepris sans que les habitants en soient informés et puissent donner leur avis par diverses procédures comme l'affichage du projet dans un lieu public. L'étude d'impact est à la disposition du public, qui peut discuter des modifications qu'il désire voir apporter au projet par le canal de procédures normales, et en fonction des dispositions légales ou règlementaires fixant les règles de protection de l'environnement dans chaque secteur de l'activité économique, ainsi que les compensations à verser à tous ceux qui, directement frappés par les mesures qui vont être prises, ne peuvent les annuler ou les modifier.

Dans les pays sous-développés, mais en développement rapide, comme le sont souvent ceux dans lesquels des actions de coopération technique sont engagées, la situation est différente. Priorité est donnée aux actions de développement ayant une rentabilité aussi grande, aussi rapide que possible. L'abondance de l'espace et parfois des ressources naturelles fait souvent ignorer des mesures qui pourraient ralentir l'exécution et finalement l'exploitation de projets de développement. Des impératifs économiques, comme une production rapide d'énergie ou une expansion des cultures commerciabilisables, peuvent faire passer au second plan des dispositions qui, à long terme sont nécessaires à l'équilibre national, mais peuvent apparaître à court terme comme conservatrices.

Mais lorsque personne ne se soucie réellemnt des impacts présents ou futurs du projet sur les milieux naturels et humains, il appartient aux organismes de coopération technique ou de financement de prendre les initiatives nécessaires; en effet ils ne sont qu'indirectement liés aux impératifs de rentabilité immédiate de l'ouvrage et autres soucis du Maître de l'ouvrage, mais ils doivent avoir en plus une "dimension supplémentaire" dans leurs objectifs, celle de l'intérêt général du pays considéré, toute action d'envergure devant être prévue dans toutes ses conséquences. Ils sont d'ailleurs parfois obligés de le faire par des dispositions législatives venant de leur propre pays; c'est le cas de l'USAID qui doit se conformer, en vertu de dispositions américaines, à des procédures impliquant une évaluation de l'environnement dans les projets qu'elle prépare dans les pays en développement.

Aussi nombre d'organismes de financement comme la Banque Mondiale, la Banque Interaméricaine ou la Banque asienne de développement incluent-ils dans le coût des ouvrages qu'ils vont financer celui des études d'impact.

Celles-ci sont parfois difficiles et coûteuses, en ressources financières, en matériel, en techniciens et les crédits nécessaires doivent être prévus dès le début du projet. Mais on a vu qu'il est souvent difficile d'en prévoir l'étendue, de discerner des problèmes encore cachés, et par suite de chiffrer le montant de ces crédits. Aussi est-il souhaitable de faire des prévisions (évaluées à un certain pourcentage du montant total du projet - 5 à 10 % par exemple) pour le règlement des études et le financement, des problèmes qu'elles vont faire apparaître.

Importance de la législation sur l'environnement

La plupart des pays développés ont conçu des législations sur l'environnement, qui ont un caractère à la fois général, car elles s'appliquent à toutes les formes de perturbation de l'environnement, aussi bien industriel qu'agricole, et à toutes les industries, et spécifique, car des règlements ou des circulaires précisent les dispositions qui doivent être prises à l'occasion de chaque réalisation importante en matière d'environnement. C'est le cas aux Etats-Unis, au Japon, dans tous les pays de la Communauté Economique Européenne, (où il existe une directive communautaire à ce sujet). C'est aussi le cas dans un certain nombre de pays tropicaux : Brésil, Inde, Indonésie, Thailande, Philippines, etc...

Ces législations ont un caractère général, lequel s'applique naturellement aux grands aménagements hydrauliques. Dans la pratique, il faut distinguer :

- les pays qui possèdent une législation et une règlementation précises sur l'état de l'environnement, ainsi que les moyens de les appliquer, les études d'impact étant partie de cette règlementation

- les pays qui ont une telle législation sans avoir réellement les moyens de l'appliquer, faute de personnel ou de ressources. C'est alors que les organismes de financement doivent intervenir pour se substituer à la carence des services officiels.
- les pays qui n'ont pas de législation sur l'environnement. Il devrait appartenir aux organismes internationaux de leur en proposer une, aussi simple et aussi réaliste que possible, car il faut pouvoir l'appliquer et donc avoir les ressources nécessaires.

La plupart des législations prévoient des études d'impact et proposent parfois des listes de points à examiner qui se retrouvent à travers les matrices mentionnées plus haut.
Dans le cas particulier des aménagements hydrauliques en pays tropical, des problèmes comparables se retrouvent dans la pluspart de ces aménagements, quelque soit le pays dans lequel ils sont établis et il est possible de dresser des listes de points à étudier assez détaillées; cependant les situations varient suivant le type d'aménagement et l'étude d'impact d'un grand barrage est naturellement bien différente de celle d'un réseau d'irrigation ou d'un forage.

La démarche intellectuelle et imaginative

Dans les pays industrialisés, les études d'impact obéissent à des règles relativement rigides fixées par la législation. Ce n'est souvent pas le cas dans les pays tropicaux en développement, et la variété des problèmes oblige l'écologiste à les imager. Sous cet angle, l'imagination rejoint le réalisme dans une conception globale de l'évolution dynamique de l'aménagement. On peut alors :
- examiner à fond la situation primitive (avant l'aménagement), secteur par secteur et en supposant qu'il existe alors un équilibre écologique dans la région (Année "Moins Un"). D'une façon arbitraire on suppose ici que l'établissment d'un grand aménagement hydraulique en pays tropical prend une dizaine d'années, depuis l'année "Moins Un" (préparation sans influence sensible sur l'écologie de la région) jusqu'à l'an "Plus Dix", au cours duquel l'aménagement est supposé fonctionner à un régime normal, les équilibres écologiques généraux étant alors établis et stables.
- imaginer quelle pourra être la situation de la région après quelques années de fonctionnemnt de l'aménagement, lorsqu'un nouvel équilibre aura pu s'établir (Année "Plus Dix").
- essayer de discerner les secteurs importants dans cette analyse globale, et en suivre l'évolution à travers les étapes d'installation de l'aménagement,
- examiner lors de chacune de ces étapes les nécessités d'une coordination entre secteurs
- s'efforcer de minimiser les effets négatifs des problèmes tout en recherchant ceux qui offrent des possibilités de développement positif (par exemple : la création d'un lac de retenue entraîne une diminution des surfaces cultivables, mais ouvre des possibilités de pisciculture).

Les différents acteurs

Une fois la décision prise, et sur le principe de l'établissement de l'aménagement, et sur sa nature, et son emplacement, ainsi que sur le choix entre les différentes alternatives possibles, une partie compliquée va s'établir entre différents "acteurs" en matière d'environnement :
- Le gouvernement représente en principe l'intérêt et est le gardien de la loi national du pays. A chaque stade de réalisation, ses représentants à l'échelon local, régional ou national, doivent pouvoir intervenir pour apprécier et si besoin rectifier ou dynamiser le projet en cours.
- Le Maître de l'ouvrage est responsable de la réalisation et de l'exploitation de l'ouvrage à tous les stades, du financement aux problèmes techniques et, en général des répercussions de l'établissement de l'aménagement sur l'environnement. Ce peut-être, par exemple, un département ministériel, la Société Nationale d'Electricité, une société régionale de développement, une commune, ou simplement un agriculteur qui veut établir un lac collinaire pour retenir son eau de pluie.
- Le Maître d'oeuvre est chargé par le Maître de l'ouvrage de éaliser l'aménagement dans le cadre du financement qui lui est imposé. Il doit faire les études, et recherches nécessaires, puis les adjudications et la surveillance des travaux, et établir les garanties et en être responsable. Il a souvent été chargé des études de base du projet.
- L'organisme de financement a un rôle complexe et parfois ambigu. D'une part il doit assurer le financement de l'ouvrage. Mais les conditions qu'il pose correspondent souvent à un réexamen des études techniques pour vérifier ou refonte plus ou moins complète suivant son jugement propre.
- Enfin l'administration locale souvent puissante a un rôle déterminant lorsqu'elle est compétente. Les services locaux : agriculture, forêts, pêche, emploi, industrie, migrations, cadastre, finances manquent souvent de moyens, mais ont le mérite d'exister. Les problèmes d'environnement seront souvent traités par eux, sous réserve qu'ils puissent avoir les ressources techniques et financières nécessaires.

Le sort des gens et l'opinion publique

Mais qui va donc élever la voix pour indiquer les conséquences désastreuses ou bénéfiques d'un projet sur le plan local et la vie des gens environs ? Comment "l'opinion publique" va-t-elle faire connaître ses suggestions, ses exigences, voire son opposition au projet ? Il faut bien dire que, dans nombre de pays, la voix timide de quelques villages qui vont être noyés ou de quelques pasteurs qui vont perdre leurs terrains de parcours n'a guère de poids face au groupe multiforme des différents acteurs ci-dessus. Tout dépend naturellement du poids politique des structures politiques locales : conseils élus à divers degrés, partis politiques, personnalités choisies suivant des formes traditionnelles (chef de villages). Mais le réalisme oblige à admettre que :
- la décision de l'aménagement est prise souvent au niveau gouvernemental et les objections locales ont peu de poids devant "l'intérêt national";
- l'absence de culture générale et technique des responsables locaux les empêche de porter des jugements valables face aux techniciens.

- dans la mesure où le budget spécial incorporé dans le budget général peut couvrir des dépenses correspondant aux dommages causés à l'environnement local, des demandes de compensation présentées par des particuliers ou des collectivités sont fréquemment acceptés.
- finalement, les intérêts locaux trouvent souvent un support auprès des organismes de financement ou de coopération technique.

Une coordination dynamique

Les études d'impact aboutissent à des recommandations de caractère pratique qui impliquent des choix, des décisions, des programmes, des actions et des financements à trouver. Tout ceci se passe dans une région déterminée, dotée de services techniques plus ou moins structurés et efficaces, mais qui, en principe devraient être responsables des problèmes sectoriels qui se posent comme conséquence du projet.

Dans nombre de pays en développement, la pratique démontre que, s'ils ne sont pas dotés de moyens exceptionnels, ces services ne sont guère en mesure d'affronter les nouveaux problèmes posés par l'existence d'un projet important. D'autre part les coordinations sectorielles indispensables ne sont guère assurées, faute d'un mécanisme suffisament dynamique pour le faire.

Il est alors indispensable qu'une double action, de promotion et de coordination puisse être exercée, dans le temps pendant la durée de la mise en oeuvre du projet, et dans l'espace, (à travers la région qui va supporter le projet).

La coordination s'exercera à chaque étape importante de la réalisation de l'aménagement par une information mutuelle des services spécialisés, une discussion libre impliquant des arbitrages et l'attribution de ressources supplémentaires aux services concernés.

La promotion de l'action sur l'environnement constituera le centre des activités dans ce domaine à travers une vue prospective des problèmes qui vont surgir avec l'avencement du projet.

Ces deux actions étroitement liées, nécessitant des décisions pratiques importantes, un budget substantiel, des arbitrages difficiles sont finalement du ressort des Pouvoirs Publics. A cette fin, une personnalité, dotée d'un statut administratif convenable et de moyens suffisants pour obtenir des services locaux, une coopération efficace, sera nommée dès le début du projet. Le "Directeur de l'environnement devrait devenir le responsable d'une "Autorité", organisme semi-autonome, s'appuyant sur un "conseil" regroupant l'ensemble des "acteurs" mentionnés plus haut et présidé par un représentant qualifié de la population.

C O N C L U S I O N

L'effort d'équipement dans lequel le monde est entré ne doit pas se faire dépens des populations locales, ou de la qualité de la vie de l'ensemble des gens. Dans quelques pays un certain degré de saturation en matière de grands travaux hydrauliques a été atteint, dans d'autres la planification est insuffisante. Mais, surtout, le temps où le choix et les désirs, voire l'existence de la population étaient oubliés lors d'un grand aménagement semble bien dépassé. Il appartient aux gouvernements, et aux organismes de coopération technique et de financement de consacrer une certaine proportion de leurs budgets d'aménagement aux problèmes d'environnement qu'ils auront décelé à travers les études d'impact.

Les actions de promotion et de coordination correspondantes seront plus ou moins autonomes, agissant naturellement en étroite liaison avec le Maître de l'ouvrage et les pouvoirs publics.

Ce sera sans doute le moyen le plus efficace de sauvegarder, à travers les vicissitudes d'une modernisation accélérée, les structures vivantes et la qualité de la vie des gens.

Quelques références

Les références sur le problème général des relations entre Environnement, Développement et Etudes d'Impact sont nombreuses, surtout en langue anglaise. Voici quelques textes utiles sur le sujet :

- Commission Internationale des Grands Barrages (1980) : Les barrages et l'environnement (Matrice détaillée).
- Ministère de l'Environnement et du Cadre de vie, Recherche Environnnement : Plusieurs publications, par exemple (1977) : Prise en compte de l'Environnement dans les procédures d'aménagement. Paris - Dasman R. (And others) (1973) : Ecological principles for Economic development.
6 FAO, Rome (1980 ....) : "Environment Papers".
- IIED (International Institute for Environment and Development), Washington D.C. Publications diverses telles que (1981) : Legal, Regulatory and Institutional Aspects for Environmental and Natural Resource Management in Developing countries, Natural Resources project.
- Union Internationale pour la Conservation de la Nature et de ses resources, (1980) Stratégie mondiale de la Conservation.
- Conservation for development Center, IUCN, (1983) Status and Application of Environmental Impact Assessment for development, J. Horberry UNEP (United Nations Environment Programme) : Publications variées par exemple : UNEP Studies on Cost-Benefit Analysis in Environmental Decision Making, Y. Ahmad (1981).
- World Bank : Publications variées par "The Office of Environmental Affaires", par exemple : (1981) "Environmental Guidelines".

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DOMESTIC RURAL WATER-SUPPLY IN AL BAYDA PROVINCE, YEMEN ARAB REPUBLIC

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ABSTRACT

The Rada Integrated Rural Development Project (RIRDP) is involved in the improvement of domestic rural water-supply and the improvement of rural sanitation systems in Al Bayda Province of the Yemen Arab Republic. The RIRDP has been operating in the area around Rada town since 1977. At first, success was limited mainly due to lack of confidence of the rural population and an approach which they did not accept. A change in approach in 1979, which included the implementation of entire water-supply schemes, increased the interest of the villagers greatly.

Experience with these schemes shows that water consumption increases after piped water-supplies are provided. A survey carried out in 1983 indicated that the average domestic water consumption will probably increase from the present 45 litres/head per day to 100 litres/head per day in the year 2000. Water fetching, traditionally done by the women, will be strongly reduced, as the vast majority of the population seem to prefer a piped water-supply system with house connections. Most water-supply systems in Al Bayda Province will consist of a groundwater source, pumping plant, rising main, reservoir and gravity-fed distribution system.

Implementation of these water-supply schemes will lead to increased flows from the houses, and thus require improvement in the sanitation systems in the area. The RIRDP has taken the initiative for planning and implementing pilot sanitation schemes. In doing so the RIRDP expects to raise the interest of the rural population in these utilities, and to demonstrate how safe health conditions can be created. Once the rural population has accepted the sanitation systems, one of the final aims of the RIRDP will be to implement both water-supply and sanitation systems in the project area.

Keywords: Yemen Arab Republic, water-supply, domestic water consumption, livestock water consumption, school water use, mosque water use, groundwater sources, rural sanitation, health conditions, sanitation development, pilot sanitation projects.

INTRODUCTION

Al Bayda Province lies in the south-eastern part of the Yemen Arab Republic in a mountainous area of approximately 11 000 km^2. It has a resident population of about 268 000 (1981 census), and its economy is based on, mainly rainfed, agriculture. The area is arid to semi-arid, with an average annual rainfall of 200 to 350 mm. Surface water resources are very limited, there being no rivers, and only a few small perennial streams in the south of Rada District. Small dams are found throughout the area, many however having silted up so that only a few are still in operation. Traditionally, water for the supply of drinking-water and for irrigation has come from numerous dug wells and a number of springs. In recent years, there has been a rapid increase in groundwater exploitation using deep boreholes.

Since 1977, the Rada Integrated Rural Development Project (RIRDP) has been active in the area, undertaking for instance, feeder road construction, domestic water-supply projects, livestock and agricultural development, and women's participation programmes. The project is financed jointly by the Governments of the Yemen Arab Republic and the Netherlands. Up to 1983 the project covered only the district of Rada but today it deals with the entire Al Bayda Province. The general objective of the RIRDP is to improve the quality of life of the rural population in the province of Al Bayda, including upgrading of the domestic rural water-supply, and of the waste-water disposal facilities in the villages.

Initially, the Water Supply Section of the RIRDP provided aid to the villages through funding, technical and organizational assistance, and - sometimes -commodities. From 1979, however, the aid has involved the provision of complete water-supply schemes partly funded by the villagers themselves. In 1984 the RIRDP Water Supply Section extended the number of activities in which it was active to include development of sanitation works.

IMPLEMENTATION

Three phases can be distinguished in the implementation of domestic rural water-supply schemes since 1977: an initiatory phase from 1977 to 1979, in which inventories and geophysical surveys were the most important; a second phase from 1979 to 1984, in which the first schemes were actually carried out; and a third phase, starting in 1984, in which both management and implementation of water-supply and sanitation will have priority.

First phase activities

The initial years of the work were characterized by inventories to provide data on water-supply in the project area and by geophysical surveys to establish which sources of water were feasible and the locations of water resources for villages with shortages. The inventories indicated that villagers were willing to co-operate and provide labour, and that skilled labour was available in the larger villages. Some villages which already had a supply, requested additional water for agriculture.

In co-operation with the Local Development Association (LDA), a number of villages were then selected in which to implement water-supply systems. Existing wells owned by landowners in those villages were already in use for irrigation, and the owners were hardly prepared to provide water for domestic water-supply. This made the implementation of commonly-owned water-supply schemes difficult.

In 1979, the RIRDP strategy with regard to the implementation had to be changed because the way in which aid was being supplied was not in line with the expectations of the rural population. Until then, the project had provided the following assistance:

- technical/organizational aid with the construction of water-supply schemes;
- co-operation in digging wells, and constructing washing basins and cattle troughs, partly funded by the Dutch Government;
- drilling of a few bore-holes and supply of pumps for villages with no shallow well or surface water sources, partly funded by the Dutch Government.

The villagers were mainly critical about the fact that mainly simple shallow wells were provided for village water-supplies. They also had a general preference for borehole supplies. In any event, adaptation of the procedures was obviously necessary to gain more support for the programme.

Second phase activities

The second phase of this work was initiated after a Dutch-Yemeni agreement which resulted in the provision of entire water-supply schemes, funded by the villagers, the Dutch Government and the Yemeni Government. This adapted strategy did not result in an immediate change in the attitude of the villagers towards the RIRDP. They still had to contribute 30 % of the total construction cost, and had not yet developed confidence in the programme. Only once the RIRDP had decreased the village contribution to 20 %, by providing labour and construction materials, did this situation improve. A 'letter of intent' for implementation was also introduced, in which total costs, scope of work and required village contribution were indicated.

The first borehole drilling programme was established in this phase, supported by geophysical surveys, at first in villages near the completed asphalt road from Dahmar to Rada and Al Bayda, and later on in previously remote areas as feeder/village roads constructed by the RIRDP were completed. Increasing popularity of the programme allowed the village contribution to be raised to 30 % of the total construction. Not only were the number of requests unaffected by this, in fact they increased. Although civil unrest stopped the activities temporarily in 1982, the workload has since reached its previous levels, while village co-operativeness remains good.

Third phase activities

In the immediate future, more attention will be paid to management of completed water-supply and sanitation schemes. This involves such things as:

- introduction of operation/maintenance manuals and schedules;
- introduction of cost/benefit plans;
- training of operators;
- survey of existing water-supply schemes.

Initially the villagers will manage the water-supply schemes themselves, but in the long term it is expected that some kind of overall organization will take over the management. This could be the Ministry of Public Works or a village organization such as the LDA.

Present procedures

The procedures which are at present in force are initiated by the registration of a village at the RIRDP, by means of an official letter signed by the village, the Local Development Association, and the court. The village must also appoint a representative to negotiate with the RIRDP on its behalf in case of questions or problems and to sign agreements between the village and the RIRDP.

Soon after the receipt of the letter of request an inventory visit is organized by the RIRDP; data collected include: number of houses, number of inhabitants, number of emigrants, quality of water from existing source, amount of water-shortage, co-operativeness, general background information. Equipped with this information the geo-hydrologist visits the village to select a site for the borehole or shallow well (i.e. if adequate surface water source is not available) and determine the location, approximate depth, and the chance of sufficient water availability. A survey is then carried out to determine the location of the storage reservoir and the pipeline layout.

Soon after the survey, the various parts of the water scheme are designed, and the village is informed of the total estimated construction cost, the expected construction activities and the required village contribution. Once the remittances from the village and/or LDA, or from village contributions in kind cover the required contribution, tender procedures are begun. The RIRDP has established tender procedures for each part of the scheme. Breaking down the entire scheme into units in this way is necessary as local contractors in the area cannot afford to supply high bank guarantees.

Contractors are then appointed for the various parts of the scheme, and after completion of construction, a "letter of completion" is sent to the village to mark the handing-over of the works and responsibility for the scheme to the village. The procedure described is stopped if a village shows a lack of co-operation by not remitting the required contribution.

WATER-SUPPLY

Water sources

There are no rivers in Al Bayda Province apart from a number of perennial stream south of Rada. Surface water is mainly runoff from slopes, occuring after rainstorms, which flows in the wadis for a few hours to several days. There are many small dams in wadis in the area, but the water from the reservoirs created after rainstorms is only used for irrigation (quantities and water quality are too low and vary too much for a reliable water-supply system). Therefore groundwater is the only suitable source of water for domestic supply in Al Bayda Province.

The type of groundwater source and the distribution over the province has been given in Table 1. As can be seen, shallow wells are widely used although boreholes are increasingly preferred in Rada basin where drilling rigs can now reach previously inaccessible areas as a result of the Dahmar-Rada-Al Bayda asphalt road, the RIRDP feeder roads and village dust roads.

It is envisaged that the number of shallow wells used for water-supply systems will be reduced in future as several shallow wells have recently dried up or have become saline (mainly caused by extensive irrigation together with the high evaporation). The salinated wells often occur in the downstream part of the wadi systems. It is therefore likely that more and more boreholes will be drilled for water-supply systems in the near future, thus avoiding this risk.

Table 1 - Types of water source in ten selected areas of Al Bayda Province (%)

	Borehole	Shallow Well	Spring Stream	Cistern
1. Rada Basin	43	47	0	10
2. Sabah/Ar Riashiyah	0	67	17	16
3. Al Bayda N-E	10	90	0	0
4. Al Bayda W	0	100	0	0
5. Wadi Juban	0	100	0	0
6. Wadi Mansur/Wadi Amad	7	93	0	0
7. Wadi Matar/Wadi Ar Rin	0	100	0	0
8. Wadi Hubabah/Wadi Ar Riashiyah	0	67	33	0
9. Wadi Dhi Na'im	0	100	0	0
10. As Sawadiyah/Abbas/Villages along Rada-Al Bayda road	0	100	0	0

Source: Ilaco (1984): Study into water resources in Al Bayda Province, Volume I - Main report.

Water-supply system

Yemeni women traditionally care for domestic water supply, fetching water from springs, hand-dug wells or cisterns. They carry the refilled jerrycans themselves or load donkeys with rubber sacks. Now that migrated Yemenis have retuned home and remote areas are being opened up, the vast majority of the rural population in Al Bayda Province aspire to a water-supply system with house connections. This generally means: a pump and diesel-driven engine, a rising main, a ground-level storage reservoir, and a gravity-fed distribution system with house connections.

One feature of the Yemeni systems is the use of unburied pipes for rising and distribution mains due the fact that nearly all the villages in Al Bayda Province have been built on rocky sites. Pipes are only buried at crossings.

It is also striking that every village aspires to its own water-supply system in order to remain independent from surrounding villages. Even if they lie only several hundred metres apart, villages will insist on their independency. This is also reflected on a smaller scale in their preference for house connections instead of having to depend on public taps. This means that in the whole province only small village water schemes exist; there are no large areal schemes at present.

The appearance of the schemes proves that management is rather poor and should be improved. The presence of the RIRDP in the area will lead to management of the systems being improved through training courses for operators, technical assistance with the operation and maintenance of the schemes, and production of management plans. The ultimate aim is that in future the villages will unite in constructing and managing their own water-supply schemes.

Water consumption

Apart from irrigation, water is mainly used by households, schools, mosques and for livestock. The per-capita water consumption of some areas in the province differs greatly from that in other areas, so no average figures can be given. The differences can be explained by the varying level of development of the respective areas. The present water consumption of a number of areas is listed in Table 2.

Table 2 - Per capita domestic water consumption in various areas of Al Bayda Province

	Domestic water consumption (l/cap. per day)	% of villages with water-supply system
1. Rada Basin	51.5	50
2. Sabah/Ar Riashiyah	56.2	6
3. Al Bayda N-E	86.7	95
4. Al Bayda W	69.1	78
5. Wadi Juban	39.5	8
6. Wadi Mansur/Wadi Amad	58.6	7
7. Wadi Matar/Wadi Ar Rin	58.8	17
8. Wadi Hubabah/Wadi Ar Riashiyah	45.9	18
9. Wadi Dhi Na'im	76.4	64
10. As Sawadiyah/Abbas/Villages along Rada-Al Bayda road	54.4	6

Source: Ilaco (1984): Study into water resources in Al Bayda Province, Volume I - Main report.

If the water consumption figures are plotted against the percentage of villages with a water scheme, it can be seen (Figure 1) that the domestic water consumption per area is clearly related to the percentage of villages with water supply in a given area. From the same curve it can be derived that the average water consumption for fetched water supplies tends to 45 l/capita per day. This value is rather high in comparison with those mentioned in the literature, which may be due to the fact the donkeys are often the water carriers. In the area with the highest average domestic water consumption and the highest percentage of villages with water-supply (Al Bayda N-E), activities to implement water-supply schemes were started as early as 1965. If the growth rate for water consumption in Al Bayda N-E is used for all the other significant areas of the province, it can be seen that by the year 2000 all domestic water consumption figures tend towards 100 l/capita per day. Consumption of water by livestock forms the next highest use of water in the Province (see Table 3).

FIG.1 VARIATION OF PER CAPITA WATER CONSUMPTION WITH % OF VILLAGES WITH WATER-SUPPLY SYSTEMS

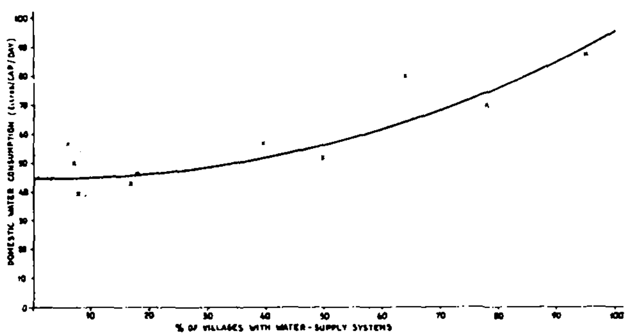


Table 3 - Livestock water consumption per head (l/day)

Cow, bull	25
Sheep	4.4
Goat	8.8
Donkey	8.8
Camel	25

Source: Ilaco (1984, Study into water resources in Al Bayda Province, Volume I - Main report.

Future livestock water consumption is assumed to be the same as present consumption as it is not considered likely that the number of livestock will increase markedly, given the present state of rangeland and the quality of the present livestock population.

Another important water user is the mosque, where according to the moslem tradition, one must wash before entering. At present water

consumption is approximately 1-2 litres per visitor each day, a figure which will surely increase after the implementation of piped schemes in the villages.
The last important group of users is formed by the schools, which have increased in number significantly during recent years. Most of the schools have been equipped with flushed toilets, often however without connection to a water-supply scheme. This implies that the present water consumption is on the low side (7-8 litres per pupil per day) due to the inadequate facilities; the normal consumption will amounts to 15 litres per pupil per day.

SANITATION

General

Implementation of water-supply schemes in general improves health conditions in a village. However, the growing waste-water production, which accompanies the increased water consumption, may nullify the improvements in health conditions. Existing sanitation systems are inadequate for the increased waste-water flows, originating from houses connected to a piped water-supply scheme.

Existing sanitation practices

Throughout Al Bayda Province there is little variation in existing sanitation practices. The traditional (balladi) system, illustrated in Figure 2, is widespread, as Table 4 indicates. Liquid and solid excreta are separated: urine leaves the toilet (located on the first floor of the house), via a gutter in the floor, through an outlet set into the wall, and drops outside onto the ground; faeces drop through a hole in the floor into a pit beneath where they decompose. All other waste-water flows leave the houses through outlet pipes in the walls from the first and second floor. Flush toilets are found only in villages with an adequate water-supply scheme.

FIG.2 TYPICAL BALLADI TOILET

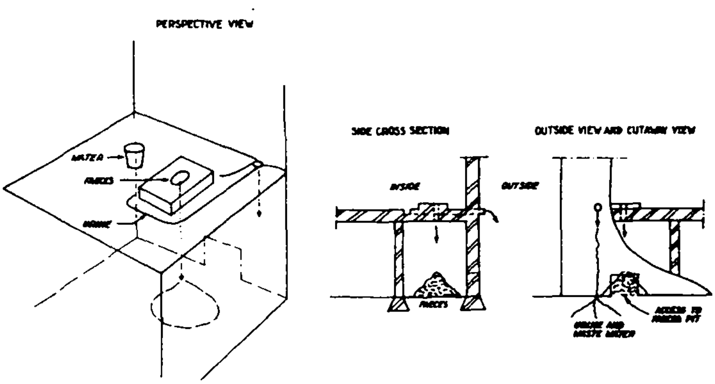


Table 4 - Sanitation systems in use in Al Bayda Province

System	Waste-water %	Liquid excreta %	Solid excreta %
Balladiyah	0	0	89
Thrown on ground/street	80	78	1
Drained to ground/street	16	17	0
Pit latrine	0	1	2
Pour-flush latrine	0	0	3
Flush toilet-septic tank	3	3	3
Flush toilet-sewerage system	1	0	1
Other	0	1	1

Source: Ilaco (1984): Study into water resources in Al Bayda Province, Volume II - Main report.

Schools and mosques are also equiped with toilet facilities. However, most of the flush toilets in the recently-constructed schools do not work, either because there is no piped water-supply or because they are not used or maintained correctly. Mosques have facilities for disposal of liquid excreta or both liquid and solid excreta, but the washing basins used before entering the mosque are often heavily polluted as they are usually not refreshed and have no drains.

Few villages have a waste-water and sewerage disposal system; those encountered are located near Al Bayda, and vary from individual systems to sewerage transporting to commonly-owned soakways.

There is hardly any development of operation, maintenance and organization/ management, except for Rada and Al Bayda towns in which sanitarians are (theoretically) responsible for rural sanitation.

Existing health conditions

Health conditions in Al Bayda Province are poor. Many of the diseases are water-related, e.g. helminthic diseases, dysentry, typhoid, protozoal infections, and - in the scarce areas with surface water - bilharziasis and malaria. They are caused by the lack of personal hygiene and environmental pollution due to:

- waste-water and garbage disposal around the houses;
- polluted drinking-water from uncovered and unprotected water sources caused by excreta from birds and donkeys;
- polluted drinking-water from inadequate distribution systems and reservoirs;
- washing basins at the mosques without proper refreshment and outlet facilities;
- free running cattle and dogs around the houses;

- the compartments of the balladiyah toilets, which are often open to the air, and inadequate emptying procedures;
- the location of the stable on the ground floor of the houses.

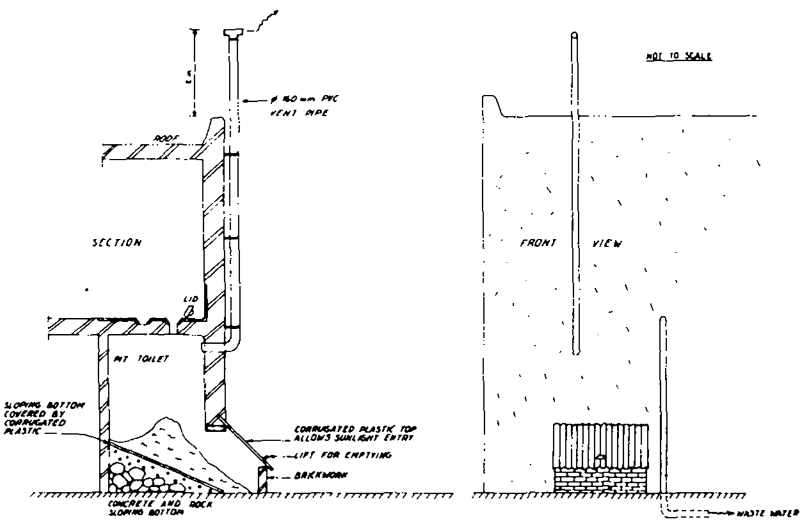
The existing health situation can only be expected to improve if potable water and sanitation are provided concurrently, and if an effective programme of hygiene education is organized to make the people aware that their health conditions are effected by polluted environment and low standards of water-supply and sanitation.

Sanitation development

In considering the economic and technical aspects of the known sanitation systems, it appeared that for the Yemeni situation in Al Bayda Province pour flush or compost toilets were suitable. Given the prevailing social customs, however, only composting toilets are justified: they are appropriate to the traditional practices, do not require water (except for anal cleansing) or expensive sewerage systems, do not cause an effluent problem, are easy in operation and maintenance and require little institutional infrastructure. They can also be built with local labour, and produce fertilizer with an economic value.

Rather than introduce a totally new type of toilet, therefore, it is better to modify the existing "balladi" type in the way shown in Figure 3.

FIG 3 TYPICAL MODIFICATION TO BALLADI TOILET



In addition to improving the domestic sanitation facilities, attention should be given to the existing sanitation facilities at the mosques. As previously indicated the ritual washing in the basins forms a health risk and therefore provision of taps or showers, toilets and adequate disposal facilities is required.

One problem which may arise in future as a result of the development of sanitation systems is groundwater pollution, especially if soakaways, stabilization ponds and effluent reuse are envisaged. The shallow wells in the province will have to be protected. Several control measures are possible, for example, reduction of hydraulic load to sanitation systems, a distance of at least 15 m between sanitation systems and water sources, lining of oxidation ponds, etc.

The Rada Integrated Rural Development Project is playing an important role in promoting sanitation development in the Al Bayda Province by implementing pilot sanitation schemes in 4 villages in 1984 and 1985, and at the same time starting up a health education programme.

If the pilot sanitation schemes become successful in the RIRDP area, the project intends after 1985 to implement water-supply and sanitation schemes concurrently.

CONCLUSIONS

- Implementation of water-supply and sanitation schemes in rural Al Bayda Province requires the acceptance, confidence and co-operativeness of the rural population.
- The risk that shallow wells become salinated or dry up is higher since irrigation will be intensified in future. Therefore, it is expected that boreholes will gradually be used more and more as the source for water-supply schemes.
- After implementation of a piped water-supply scheme, water consumption will increase. In Al Bayda Province the domestic water consumption is expected to increase from the present level of 60 l/capita per day to 100 l/capita per day in the year 2000.
- The expected waste-water production under these changed circumstances cannot be disposed of safely through the existing sanitation systems. These existing systems will therefore need to be upgraded or new sanitation systems will need to be implemented, in order to cope with the expected quantities of waste-water in the future.
- The majority of the rural population of Al Bayda Province is at present unaware of the unhealthy environmental conditions caused by garbage and waste-water disposal around the houses. However, it is expected that the pilot sanitation projects initiated by the RIRDP will improve this situation.

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WATER QUALITY MANAGEMENT IN DEVELOPING COUNTRIES PART 1

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Abstract

It may be expected that over the next decade the management of water quality problems will be one of the outstanding issues relating to the protection and conservation of the national stock of water in each country. In the past, particularly in countries well endowed with water resources, this has been considered to be a relatively negligible problem; however, the rapid aggregation of population in major urban centres, the polarization of industries, and the heavy dependence on chemical products in the agricultural sector are leading to a serious deterioration of water quality in developing countries.

The paper will review the nature of the pollution issue, the institutional requirements to deal with the problem in an effective and comprehensive manner and the near term actions which governments should take to protect their existing water resources for the generations to come.

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WATER QUALITY MANAGEMENT IN DEVELOPING COUNTRIES

Introduction

It may be expected that over the next decade the management of water quality problems will be one of the outstanding issues relating to the protection and conservation of the national stock of water in each country. In the past, particularly in countries well endowed with water resources, this has been considered to be a relatively negligible problem; however, the rapid aggregation of population in major urban centres, the polarization of industries, and the heavy dependence on chemical products, particularly in the agricultural sector, are leading to a serious deterioration of water quality in developing countries.

Due to the high priority given to economic growth, industrialization and budgetary controls in developing countries, water quality protection had been almost universally deferred in those countries. Today however, there is widespread realization that water quality management is an economic imperative for nations as they face increasing and often competitive demands on the use of this important resource. The attainment and maintenance of appropriate water quality necessary to sustain the desired uses of water bodies is vital to the development of nations. To protect adequately water quality, governments will need to understand the technological aspects of pollution problems, and will need to respond with effective policies and institutions.

This paper reviews the nature of the pollution issue, the institutional requirements to deal with the problem in an effective and comprehensive manner, and the near term actions which governments should take to protect their existing water resources for present uses and for generations to come. In particular, major sources of water pollution typical in developing countries are presented and discussed, and the effects of water-borne pollutants are reviewed. The importance of establishing legal authority for water quality management activities is discussed as well as various policy instruments available to effect pollution abatement. Important tasks for governments to undertake in the near future are reviewed. The paper will conclude with a brief preview of the activities envisioned in this area by the United Nations Department of Technical Co-operation for Development (UN/DTCD) in the coming decade.

A. SOURCES OF POLLUTION AND ITS EFFECTS

Water bodies are becoming increasingly subject to a variety of pollution sources attributed to urbanization, industrialization, the proliferation of chemical products, and even natural sources. The term "pollution" correctly means the discharge of any material in quantities that interfere with a desired use of the water body. Implicit in this definition is the recognition that a discharge may or may not be a waste material, and that it may be either of anthropogenic origin or naturally occuring. It also recognizes the assimilative capacity of receiving waters, and implies that discharges do not constitute "pollution" until and unless the particular discharge characteristics of the water body result in concentrations of discharged material that interfere with the use of the water.

Pollutants can generally be placed into the following classifications (UNEP/WHO, 1983):

Organic Compounds
Inorganic Salts
Metals
Nutrients
Particulates
Radionuclides
Heat
Micro-organisms

The effect of any particular pollutant is a function of many factors related to the characteristics of the discharge itself as well as characteristics of the receiving waters.

Pollutant sources may be categorized as either point or non-point sources. Point sources are discrete "end of pipe" discharges such as, for example, an industrial waste-water effluent or domestic waste from a municipal sewerage system. By contrast, non-point sources exert an impact over a diffuse area. Waste characteristics, i.e. mass rates and pollutant concentrations, are more difficult to determine for non-point sources than for point sources which are more amenable to study. Table 1 (UNEP/WHO, 1983) lists many non-point sources potentially affecting marine environments. These same sources could affect fresh water bodies as well. In addition, fresh water bodies may be affected by water logging or salt water intrusion from coastal waters, estuaries, or brackish ground water. Table 1 does not include atmospheric inputs, as data were insufficientfor UNEP/WHO to assess the magnitude and the need or opportunity for control of atmospheric pollution.

It is important to note that various substances present in pollutant discharges may already be present in the receiving water at background levels. Examples are minerals and certain metals (iron is common in many locations) which may result from natural geological formations. Anthropogenic sources add to these background levels. The background levels define a lower boundary of potential water quality objectives.

Although pollution has often been assumed to be a minor problem in developing countries, it is in fact fouling both surface and ground waters in Asia, Africa and Latin America (Fano and Brewster, 1982). Untreated wastes from agro-industrial processes and effluent from new factories have destroyed fisheries, reduced available water supplies and impaired agricultural productivity (USAID, 1979). Water bodies in almost all major urban areas are polluted by untreated organic discharges from human and industrial activities.

1. Urban sources of pollution

The environmental effects of domestic wastes grow exponentially as communities grow in size from villages and towns to cities and metropolises. As population density increases, so does the spatial concentration of human and household wastes. Many urban centres in developing countries lack modern sanitation facilities for the collection and disposal of domestic wastes. This is particularly true in urban fringe areas which have been created through unplanned and uncontrolled growth due to tremendous migration of peoples from rural areas. As a result of this lack of facilities, human excrement and sullage is placed at curbsides, in ditches, or at best in open pits and dung heaps. The health risks through direct exposure to these wastes are obvious. Additionally, however, wastes disposed in this fashion result in urban runoff highly polluted with pathogens and organic materials that can have a serious impact on the water quality of nearby surface waters and shallow ground waters.

This situation is evident throughout the world but is perhaps epitomized by conditions in Mexico City whose 425 square miles already contain 15 million people. Millions of "paracaidistas", so called because they appear suddenly as if from the sky, inhabit slum areas such as Nezahaulcóyotl which began as a squatter settlement close to Mexico City's garbage dump. Here, open sewers and surface runoff after rains create rivers of sewage, which may contaminate local water supplies. Similar conditions exist in Rio de Janeiro where runoff from slum areas contributes to the degradation of Guanabara Bay and threatens even the famous Ipanema Beach. Efforts are currently underway in Cairo and Alexandria to control urban runoff through the expansion and rehabilitation of sewers. In Singapore, an ambitious programme to divert urban runoff into made-made reservoirs has just begun. The collected runoff will receive extensive treatment and will contribute to the city's water supply.

Urban centres in developing countries have also undergone a concentration of modern industry and attendant industrial water pollution during the last two decades. In Latin American countries such as Venezuela and Mexico, there has been rapid development of urban industrial complexes producing petroleum, petrochemical and steel. Oil and gas processing cities have proliferated in the Middle East and, in locations such as Yanbu, Saudi Arabia, have provided the impetus for the growth of cities. Energy, chemical, and metallurgical industries have expanded rapidly in China, as have heavy industries, such as steel processing and petrochemicals in India. The recent pattern of industrial concentration in urban areas is widespread in developing countries. Although most countries recognize the need to combat pollution, controls are generally inadequate.

2. Industrial water pollution

It is estimated that the demand for water from industrial users will increase rapidly over the next two decades in the developing countries. For example, it has been estimated that in India, industrial and other non-agricultural uses will claim 16 per cent of consumptive use in the year 2000 as compared to only 8 per cent in 1975 (Varma, 1978). If unchecked, this increase in industrial water use will naturally be accompanied by an increase in industrial water pollution.

Untreated industrial wastes can have serious impacts on receiving water bodies. The specific effects in any given case are dependent on the assimilative capacity of the water body and the characteristics of the waste. It is difficult to summarize industrial waste characteristics concisely because of the wide spectrum of industry classifications, the diversity of production technologies and modes of operation employed within each industry, and differences in raw material inputs. Nonetheless, the World Bank has studied waste streams in over 30 classes of industries and has developed guidelines for effluent limitations (1984).

The major industries in the traditional sector which are causing widespread water pollution are those which process primary products (often for export), such as sugar and oilseed mills, mineral extraction and processing facilities, coffee factories and tanneries. Agro-industries can become a major source of pollution when an increasing proportion of the population becomes involved in production of cash crops and industries begin concentrating in growing areas.

As an example, typical waste-water characteristics and effluent guidelines for a palm oil mill are shown in Table 2 (World Bank, 1984). Like other agroindustries, palm oil milling produces a waste stream high in organic content. When discharged to receiving waters, the organic materials can cause rapid oxygen depletion. In Asia, localized pollution from agro-industrial operations such as sugar and palm oil mills adversely affects water supplies and fisheries and has become a major problem in some areas. One of Malaysia's most successful economic developments has been the establishment of palm-oil plantations and processing mills, which have made Malaysia the world's major exporter of the oil. However, the effluent from the processing plants has a high organic concentration and is rarely treated, thus polluting water supplies, damaging fisheries and adversely affecting the health of people in the countryside. The total organic waste loading from palm-oil wastes in 1975 was equivalent to the total wates from a community with a population slightly larger than the population of West Malaysia (USAID, 1979). The World Bank (1984) recommends wastewater treatment for palm oil mills that would remove approximately 99 per cent of the organic load.

3. Pollution in rural and agricultural settings

Even in rural areas, far from industrial and municipal discharges, pollution of water resources may occur as a result of natural causes or man's manipulation of the environment.

Agricultural operations may have an adverse impact on the quality of water resources in several ways. One way is through drainage and runoff from fertilized cropland. Heavy organic loadings, sediments, micro-organisms, and high concentrations of nitrogen and phosphorus nutrients can be washed into low lying streams, rivers and lakes, contributing to oxygen depletion, eutrophication, and undesirable growth of aquatic plants and weeds. Pesticides may also be present in the runoff. The problem is accentuated in temperate climates where spring thaws result in unusually high loadings of short duration.

Agricultural operations may also affect receiving waters through the return flows of irrigation water. Though similar in composition to cropland runoff, irrigation return waters are typically discharged in the form of a point source. Runoff and drainage from animal feedlots are also high in organic loading, micro-organisms and nutrients. Relative to croplands, however, waste-water quantities generated from feedlots are relatively small and their impacts are relatively localized. Improved farm management practices can be introduced to reduce pollution from these sources and simultaneously improve farm productivity and conserve resources.

Forestry operations, particularly if not properly managed and controlled, can also have an impact on water quality. Logging operations can result in increased runoff, carrying sediments and nutrients to water courses. The problem has become extremely serious in some developing countries, where whole mountainsides have been virtually denuded for fuel. As a result, precious topsoil is continually being washed away. Several water supply reservoirs have been rendered useless by the deposition of sediment. In coastal areas, ecologically sensitive coral reefs have been smothered and killed. Reefs are an important resource in many Caribbean and other coastal locations because of their integral role in fish production and because of their recreational value to the tourism industry.

Contamination of water supplies by domestic wastes is perhaps the most serious water quality problem in rural areas. It has been estimated that on average from 1970 to 1980, only 29 per cent of the population in rural areas of developing countries had access to safe drinking water (United Nations, 1980b). This condition results primarily from a lack of basic sanitation. During the same ten year period, only 13 per cent of rural populations in developing countries were estimated to have access to sanitary excreta disposal. Sanitary facilities are lacking and people are unaware of the cause/effect relationships between indiscriminate excreta disposal and health. It is therefore common for excreta to be disposed of in rivers and lakes used for water supply. Another common practice is for excreta to be eliminated into fields, ditches or garbage heaps from where it can be washed by rainfall to nearby water supplies. The health effects are staggering: a major proportion of all sickness and disease (including such deadly diseases as diarrhea, polio, typhoid and others) can be attributed to inadequate water or sanitation.

Fortunately, improvements are being made on global and national levels under the stimulus of the International Drinking Water Supply and Sanitation Decade. National governments are being encouraged to devote larger allocations of resources to this sector. A great deal of emphasis is placed on the development and installation of appropriate low-cost technologies such as hand pumps and latrines. A note of concern is surfacing, however, over unanticipated incidences of groundwater contamination caused by seepage from latrines (UN/ESCAP, 1983). Remedial actions will be necessary in these instances. Future latrine installations will need to be sited with greater attention to possible effects on water supplies.

4. Effects of pollution

The effects of a pollutant discharge to a water body depend on the characteristics of the discharge and natural characteristics of the water body. Pollutants can be categorized as either conservative or non-conservative substances. Organic pollutants as well as some inorganic pollutants and many micro-organisms are degraded by natural self-purification processes within the water body. Their concentration reduces with time. Hence they are termed non-conservative. The rate of decay of these non-conservative pollutants is a function of the particular pollutant, receiving water quality, temperature and other environmental factors. Many inorganic chemicals, such as heavy metals, are not affected by natural processes, however. The concentration of these conservative substances can only be reduced by dilution (Tebbut, 1982).

Figure 1 compares the concentration of a conservative substance versus a non-conservative one over an equal stretch of river assuming constant flow throughout. As seen, while the concentration of the non-conservative substance decays with time, the concentration of the conservative substance remains constant.

Whatever the resultant concentration for a given discharge to a given body, the significance of that concentration depends on the desired use of the water. The main effects of pollution may be:

(a) Contamination of water supplies, resulting in health risks and increased load on water treatment facilities;
(b) Fish kills or decline in fish productivity;
(c) Pollution of irrigation water, posing health risks or inhibiting crop productivity;

(d) Degradation of recreational and aesthetic aspects of water;
(e) Creation of odour nuisances;
(f) Hindrance to navigation.

The relative importance of these points is a function of the desired use of the water body. Parameters important for various uses are presented in Table 3. Each use will have specific water quality requirements. Water quality standards proposed by Prescod (1974) for the most important water uses in tropical countries are presented in Table 4. Since water is often a scarce resource, it is desirable to practise multi-purpose use. Obviously, however, the water quality requirements for the various uses will not always be compatible (Tebbut, 1982). In setting water quality objectives for a particular water body, society must consider the costs of attaining the objectives versus the beneficial uses of the water. A risk-benefit approach is recommended.

Potable water supply, for example, is a vital use of many fresh water sources in both developing and industrialized countries. The levels of pathogenic organisms and toxic substances are critical for a potable water supply, and waters intended for this use must be protected from such pollutants. The degree of protection, however, as reflected by the standards adopted, is subject to risk-benefit analysis. The risk-benefit approach has recently been espoused by the World Health Organization (WHO). Earlier drinking water standards developed by WHO in 1971, have been superseded by WHO's new "Guidelines for Drinking-Water Quality" (1984). The Organization's current posture is to encourage nations to develop their own individual and appropriate standards in the context of the prevailing environmental, social and economic conditions (Thompson and Wernicke, 1984). WHO recognizes that the implementation and enforcement of standards, and the actual achievement of water quality in compliance with those standards, can be a costly proposition. Countries should weigh (qualitatively or quantitatively) the total costs of resources devoted to pollution control versus the benefits to be reaped by the investment, and should also consider the risks imposed by lesser degrees of control. As expressed by Thomann (1972), "The central question in water quality management is: 'To what level are we ... willing to invest moneys (or forego some other opportunity or live with some particular problem) to protect, enhance or otherwise utilize a particular body of water?'"

Risks and benefits are easily illustrated when considering another major use of water in developing countries: commercial and subsistence fishing. Pollution of fisheries risks declining catches and adverse health, taste and odour effects. Conversely, the benefits of pollution control include continuing or improving fish productivity and wholesomeness of this important food source. The main pollution parameters to be controlled for fisheries are dissolved oxygen, pathogenic organisms, toxic substances, temperature, non-biodegradable substances that concentrate in the food chain (such as DDT), and substances that impart undesirable taste or odour to the fish (Lohani, 1982). An example is provided by the Gulf of Thailand where pollution is endangering health and causing a serious dwindling of the levels of fish stock (World Water, 1984). According to a study carried out by the Economic and Social Commission for Asia and the Pacific (ESCAP), the Gulf is subjected to almost eight tons per day of biological oxygen demand. As a result of this pollution, fish productivity has dropped to a point where many commercial fishermen are barely able to recover their investments. Additionally, 84 per cent of shellfish sampled in an open market in Bangkok in 1979 contained bacteria likely to cause food poisoning and gastroenteritis.

The third and perhaps most important use of water in developing countries is irrigation. In India, for example, agricultural uses of water account for about 90 per cent of all water used (Varma, 1978). Because of its importance to food production, irrigability will often play a governing role in deciding pollution control strategies. Of particular concern is the protection of irrigation water from high concentrations of salinity and sodium. Salinity, as measured by electrical conductivity and total dissolved solids, causes plasmolysis of plant cells at concentrations that vary with the plant type, contact period and ambient temperature (Lohani, 1982). The sodium adsorption ratio (SAR) is also important in controlling the alkalinity of soils.

Degradation of water resources by uncontrolled pollutant discharges can therefore have significant adverse effects on important sectors of a developing nation's economy, particularly with regard to agriculture, fishing and potable water supply. Governments must develop effective policies and institutions to correct current detrimental effects of pollution and to protect water resources for future generations. A risk-benefit approach is recommended, however, so that limited resources may be allocated in a cost effective manner. The attainament or maintenance of pristine water resources is neither economically justifiable nor, in many cases, technically achievable. Rather, it is efficient for a society to define the desired uses of specific water bodies, and for water quality objectives to be formulated to provide for those uses.

B. INSTITUTIONAL REQUIREMENTS

1. Legal authority for water quality management

Unless the efforts of water quality management rest on a firm, defensible legal foundation, they are virtually certain to fall short of their objectives. While most developing countries have basic legislation empowering government agencies to control some forms of water pollution, the laws may not be totally suitable in light of prevailing political, economic and cultural realities (Fano and Brewster, 1982). For this reason, they may not be as effective as would be desirable. Similarly, many developing countries lack the institutional capabilities and technological resources to enforce pollution control laws. Enforcement considerations will be discussed later in this paper, however.

Laws on water pollution vary widely in Asia. In some countries, such as Bangladesh, Pakistan and Thailand, water pollution is covered by general statutes; in others, such as India, Indonesia,, Malaysia, the Philippines and Singapore, there is specific water quality legislation. In Latin America, a few countries, such as Colombia, Mexico and Venezuela, have comprehensive laws and dominant institutions for environmental management, including water pollution control. Most of the other countries have at least established environmental units with a primary co-ordinating function.

In most African countries, environmental laws dealing directly with water pollution are in the early stages of development, although many of the colonial governments had left general public health codes which prohibited "nuisances," such as water pollution, and authorized their regulation. Just a few years ago, however, Egypt enacted an exemplary comprehensive water pollution control law (Water Information Center, 1982). The law protects Egypt's surface and ground waters from deleterious effects of unregulated waste discharges. Under the law, a permit issued by the Ministry of Irrigation in accordance with recommendations made by the

Ministry of Health, is required for the disposal of all solid, liquid or gaseous waste. Health officials periodically test permitted discharges in order to check compliance with permit standards and conditions. Violaters are faced with severe enforcement proceedings. The law requires that new industrial, commercial, or tourist-related installations include appropriate pollution control facilities. Existing installations were given one year to arrange for waste treatment.

Having established a legal basis for water quality management activities, each country will have several policy instruments available to it for dealing with water pollution, among which are the following (Fano and Brewster, 1982):

(a) Direct charges on effluents as an incentive to polluting entities to reduce waste loads;
(b) Subsidies to promote pollution control, using tax rebates or payments to offset costs of pollution control;
(c) Government standards on effluents from production processes, limiting discharge levels of certain substances into water courses;
(d) Government licenses under which permits will only be issued to entities using "clean" processes;
(e) Requirements of environmental impact statements from potential investors of new projects.

In addition to these formal measures, informal education and persuasion may have some effect on polluting entities. Industrialists particularly may respond to economic benefits promised by waste reduction measures such as resource recovery, waste-water reuse, by-product formation, etc. The long term economic effects of degrading one of a nation's most vital resources, its water, also should not be missed.

Each government must choose for itself the appropriate combination of measures which will maximize the effectiveness of pollution control efforts. Various policy instruments available for implementation are described below.

2. Economic incentives

Various economic incentives may be applied to induce polluting entities to reduce discharges on the basis of self interest (Fano and Brewster, 1982). In a market economy, all types of resources are allocated to their most efficient use by price. Many environmental resources, water quality in particular, are unpriced and remain outside the market. Water quality may be degraded or "used up" by pollutant discharges, but this is not reflected in the price system. Such use of a resource is termed as an "externality" because the cost of its use, in terms of environmental degradation, is not borne by the user of the resource (Anderson, 1977).

Theoretically, if the costs of consuming water quality could be quantified and charged to the polluting entity, he would discharge less. An effluent charge levied by government on the quantity of pollutants in a discharge should have the same effect as the market price on the polluting entity's decision to consume water quality. This approach, known as the "polluter pays principle," internalizes to the polluting entity to some degree the cost of using an environmental resource.

a) Effluent charges

The effluent charge system involves government setting a charge at a level at which the marginal cost of pollution is slightly greater than the marginal cost of pollution control. As a result of such a charge, polluting entities are induced to initiate pollution control measures for cost-saving purposes. Economists generally consider this system to be the most effective means of reducing water pollution.

Effluent charges have been used in several European countries. Czechoslovakia provides a good example (Anderson and others, 1977). The government imposes a basic charge on biochemical oxygen demand (BOD) and suspended solids (SS), and adds a surcharge of from 10 to 100 per cent of the basic charge depending on the extent to which the concentration of BOD or SS is increased in the receiving waters. The charges are based on the costs of operating existing treatment facilities and do not reflect capital costs for new or expanded facilities. Consequently, the effect of this scheme is to induce proper operation of existing facilities, but not to induce capital investments. The Czech system does, however, allocate collected revenues to subsidies for such investments. Information on discharge levels is reported by the dischargers themselves with spot checks by government inspectors and penalties for falsification, assuring the accuracy of reports.

The generation of revenue is an important advantage of the effluent charge system. Another advantage is that it requires less information than other approaches and therefore has lower administration costs.

For developing countries considering the introduction of effluent charges, some of the following suggestions (United Nations, 1980a) regarding favourable ingredients of such a system might be useful:

(a) When an effluent charge system is introduced, initially low rates can be established with dates for specified rate increases indicated;
(b) The charges can be related to a few pollutants which are comparatively easy to measure by techniques which yield consistent results;
(c) The administration of an effluent charge system is greatly simplified by a table of pollution coefficients, establishing levels of pollution per unit of output or per employee. Provisions must be made for sampling and for basing payments on actual discharge of pollutants;
(d) An effluent charge system should emphasize regional differences, including the assimilative capacity of the water course.

b) Tax incentives

Tax reductions for investments in pollution control are another important form of economic incentive. The Environmental Code of the Philippines, enacted in 1977, allowed half of the tariff duties and compensating tax on imported pollution control equipment to be waived for a period of five years from the date of enactment (UNDP Task Force on the Human Environment, 1978). Similar rebates were available for domestically produced equipment. The Code also made available tax deductions for certain pollution control research.

Other countries offer tax incentives to new industries to site their facilities away from urban concentrations. Still others allow industries to accelerate the depreciation of treatment facilities in computing taxes.

The nature of a tax incentive system is in effect to subsidize capital investments in pollution control facilities. This may be particularly attractive in those countries where few such facilities currently exist. Tax incentives do not, however, encourage non-structural means of pollution control such as alterations in production processes and raw material inputs.

3. Pollution control standards

Direct regulation has been used in many countries, notably the United States and the United Kingdom, to achieve significant gains against environmental degradation. Using this approach, government sets maximum allowable limits on discharges for particular pollutants or industries and establishes the administrative and judicial means to enforce these standards.

a) Ambient and effluent standards

Ambient water quality standards specify the minimum conditions which must be met for specific parameters at specific locations in a water body. For example, an ambient standard for a specific river may require that dissolved oxygen, averaged over a 24-hour period at a selected river mile point, must not fall below 4 parts per million (ppm) more than one day per year. Effluent standards, on the other hand, specify the mean or maximum permissible discharge of a pollutant from a particular source.

Ambient and effluent standards together are complementary components of an approach to water quality management. In situations where numerous waste discharges exist, achieving an ambient water quality standard through independent regulation of the various effluents will be impossible. Rather, the government must co-ordinate the various effluent standards in such a way as to achieve the desired goals in the receiving water body (Kneese and Bower, 1972).

When used in developing countries, both types of standards should be set in consideration of technological feasibility and cost, as well as potential effects on human health and ecology. The standards should be related to the desired uses of the water body. Water bodies may be classified according to their designated best uses and standards developed for each classification. The scheme of classification used in India, for example, is as follows (Ranganathan, 1982):

Designated Best Use	Classification
Fresh Water	
Drinking water source without conventional treatment but after disinfection	A
Outdoor bathing	B
Drinking water source with conventional treatment followed by disinfection	C
Propagation of wildlife, fisheries	D
Irrigation, industrial cooling, and controlled waste disposal	E

Sea Water

Salt pans, shell fishing, contact water sports	SW I
Commercial fishing, recreation (non-contact)	SW II
Industrial cooling	SW III
Harbour	SW IV
Navigation, controlled waste	SW V

India's water bodies are classified according to this scheme and water quality standards appropriate for each class are applied.

One disadvantage of this system is that it does not provide an incentive to dischargers to reduce waste generation beyond that necessary to meet regulatory requirements. Enforcement is usually carried out by spot checks by government inspectors, with penalties imposed on violators. Violators may prefer to delay compliance with standards and to engage the government in long legal battles. By contrast, a system of effluent charges provides an immediate economic incentive for the polluting entity to reduce its discharges.

Another disadvantage, of particular concern to developing countries, is that the administrative and enforcement expenses involved are enormous (Anderson and others, 1977). The political and economic costs of an effective programme of direct regulation are too high for most governments to bear.

b) Mixed systems: charges and standards

Mixed systems may be viewed as either regulatory programmes, in which charges play an enforcement role, or as charge systems in which specified discharge levels have been exempted from the charge (Fano and Brewster, 1982).

Mixed systems have been enacted in the German Democratic Republic and in Hungary. Both countries levy charges on all discharges in excess of fixed effluent standards. This has the effect of providing an immediate economic incentive to reduce discharges. There is no incentive, however, to reduce discharges below that necessary to meet effluent standards.

4. Environmental impact assessment

In recent years, developing countries have become increasingly aware of the necessity for predicting the impact of a new development project, industrial or otherwise, on environmental resources, including water quality. Environmental impact assessments (EIA) have been used for this purpose in industrialized nations for more than a decade. Many developing nations have enacted legal provisions requiring EIAs from investors and developers. Many are modelled on the United States' National Environmental Policy Act (NEPA). NEPA's requirements for an EIA are extremely thorough and comprehensive. For small projects and in situations where funding is inadequate for a detailed EIA, other approaches such as "fatal flaw" and diagnostic level assessments may be more applicable (Thompson and others, 1983).

Whatever the level of detail selected for an EIA, the basic objectives of an assessment are two-fold. One is to develop information for planning and decision-making that will assure that a proposed development is compatible with its environment; the second is to determine the total cost and net overall benefit that an action will incur. This includes not only the direct financial costs and revenues arising from construction and operation, but also the broader environmental costs and benefits due to redirection of natural resources (Thompson and others, 1983). Environmental costs and benefits are frequently not quantifiable in strict monetary terms and must be judged qualitatively.

The advantage of an EIA lies in its ability to predict environmental consequences of a development project and to influence the project concept and design to avoid or reduce adverse impacts.

The concept of environmental impact assessment has been adopted by several Asian countries, notably the Philippines, India, Indonesia, Pakistan, Singapore and Sri Lanka. The Philippines has legislation empowering the National Environmental Protection Council (NEPC) to require EIAs from significant projects. Guidelines for the implementation of EIAs were formulated by NEPC, and provision was made for Philippine officials to receive training in assessment techniques.

Environmental impact assessments have been used sporadically for development projects in Mexico, Panama, Costa Rica, Haiti and Venezuela, particularly for projects in the energy sector. Though growing in popularity, their use has not become permanently established in most Latin American countries. Several training programmes in that region have been sponsored by international development assistance agencies and, in at least one case, by a professional association in a developing country (Thompson and others, 1983). In 1982, the Dominican Association of Sanitary and Environmental Engineering invited a United States consulting firm engaged in environmental studies in the Dominican Republic to conduct a seminar on environmental impact assessment for the benefit of local administrators, planners, engineers and other specialists.

In Africa, environmental impact assessments have not yet been widely introduced. The importance of such analyses has become more apparent as national and international funding agencies require consideration of environmental impact as a condition to loans or grants for development projects.

Many developing countries throughout the world have gained good experience in preparing EIAs through submitting project proposals to development assistance agencies. Although environmental considerations may increase the total cost of a project, developing countries are increasingly required to prepare EIAs in support of projects for which external financing is sought. EIAs are required by the World Bank, for example, as part of any project appraisal (Walter, 1975). As early as 1971, leading officials of the World Bank were alerted to the environmental repercussions of projects financed by that institution. The appropriate organizational changes were instituted with a view to helping the developing countries avoid some of the adverse environmental consequences of development. The general feeling was that the costs of prevention were far smaller than the costs of environmental reparation later on -- if then it would be possible at all (Walter, 1975).

Other international development assistance agencies in 1980 declared their intention of requiring appropriate environmental measures in the design and implementation of development projects (United Nations Environment Programme, 1980). International development assistance agencies have thus provided a framework within which developing countries can learn to prepare, and require of others, environmental impact statements prior to the implementation of projects.

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WATER QUALITY MANAGEMENT IN DEVELOPING COUNTRIES PART 2

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Abstract

It may be expected that over the next decade the management of water quality problems will be one of the outstanding issues relating to the protection and conservation of the national stock of water in each country. In the past, particularly in countries well endowed with water resources, this has been considered to be a relatively negligible problem; however, the rapid aggregation of population in major urban centres, the polarization of industries, and the heavy dependence on chemical products in the agricultural sector are leading to a serious deterioration of water quality in developing countries.

The paper will review the nature of the pollution issue, the institutional requirements to deal with the problem in an effective and comprehensive manner and the near term actions which governments should take to protect their existing water resources for the generations to come.

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C. ESTABLISHING A NATIONAL PROGRAMME

The institutional requirements necessary to manage the quality of a nation's water resources effectively must be carefully planned and developed. An incentive system must be planned, institutions created or existing institutions strengthened, and water quality management plans prepared. Good planning and institution-building will result in the efficient use of human and financial resources to achieve desired water quality objectives.

1. Making an incentive system work

A workable incentive system is the core of a comprehensive water quality management programme. Incentives are required in order to induce entities to reduce levels of discharges. Economic, regulatory and administrative incentives have already been discussed, as have informal incentives, such as education and persuasion. Countries currently practicing water quality management through an incentive system typically employ some mix of incentives - economic, regulatory, administrative and informal (Bower, 1982). The particular incentives to be used by a developing country are, of course, at the discretion of the country and should be chosen in the context of the existing socio-political setting.

The chosen incentives must be backed by surveillance and enforcement programmes. A surveillance programme is comprised of monitoring and inspection activities. Compliance monitoring determines whether or not a discharge conforms to permit conditions. In addition to compliance monitoring, monitoring of ambient water quality determines the extent to which desired water quality objectives are being met in the receiving water body. On-site inspection of pollution control facilities completes the surveillance programme. Inspections determine whether or not specified equipment is installed and is properly operated and controlled.

An enforcement programme is based on a set of sanctions for failure to comply with pollution control requirements (Bower, 1982). Administrative sanctions may be informal, such as warning notices or meetings, or formal, such as the issuing of orders or fees, or revocation of permits. More severe judicial penalites, such as civil penalties (fines), injunctions or criminal penalties may also be imposed.

Many developing countries lack the technical hardware and the specialized human resources necessary to carry out an adequate surveillance programme. Nations must establish laboratory facilities, appropriately equipped, for the analyses of waste waters and ambient water samples. Capabilities for storing, retrieving, and analyzing the large amounts of data so generated must also be established.

Many developing countries also lack effective judicial systems necessary to enforce legal requirements against violators. Where lacking, legal penalties must be established or existing penalties strengthened. In addition, the political will to prosecute violators must be created and maintained at every level of government.

2. Institution building

A comprehensive water quality management programme entails a variety of activities, including: data collection and analysis; research; planning; development and application of incentives; design; construction; operations and maintenance; surveillance; enforcement; and technical and financial assistance. In most cases, a number of government agencies will have responsibility for one or more of these activities (Bower, 1982). Thus, a fundamental task is deciding how to allocate these activities among the various agencies. A more complex task is building the agencies' capabilities to perform their assigned functions. Methodical studies of parallel institutions in other countries can be of significant help in this regard.

The efficacy of any institution is dependent on the abilities of the personnel charged with its operation. Water quality management, as seen from this discussion, is a complex discipline comprising elements of law, sciences, engineering, administration, economics and finance. Qualified specialists with advanced training are therefore required. Developing countries must take steps to provide for these types of education, either through indigenous centres of higher learning or by sending specialists to study abroad. The latter approach risks the "brain drain" problem common in developing countries. Safeguards against this eventuality would have to be considered.

3. Assistance to municipal and state polluters

Villages, towns and even cities typically lack the technical expertise and financial resources necessary to plan and construct pollution control facilities. Technical and financial assistance will have to be provided by national governments and outside sources if adequate control of domestic wastes is to be achieved. Financial, if not technical, assistance will also be necessary to curtail pollution from many state-owned industries.

Such assistance is frequently provided by international development assistance organizations. For example, the World Bank is currently engaged in an extensive river clean-up programme in Rio de Janeiro State in Brazil. The Bank has approved a $US302 million loan to assist in the clean up of the Paraiba do Sul River, one of the country's dirtiest rivers. The river, which supplies water to some nine million people in Rio de Janeiro, is grossly polluted with sewage and industrial wastes. One of the offenders is the State-owned National Iron and Steel Company. The Bank-financed programme aims at inducing the nearly 2,000 companies in the Paraiba Valley to install pollution control facilities, and at the expansion of sewage control facilities.

The Community Water Supply and Sanitation (CWSS) Programme in the Western Development Region of Nepal provides an example of outside assistance being channelled through regional and local institutions to improve water quality (Strauss, 1983). The primary objective of this programme is health improvement, which is to be achieved through construction of village water supply systems, latrines for schools and families and ferro-concrete tanks for water storage. With related programmes aimed at education and training in sanitation, water-borne disease should be reduced through a lower incidence of drinking water contaminated by excreta. External assistance provides a catalyst to mobilize community resources for improved water quality.

The community is supposed to be involved in all stages of the project -- design, construction, operation and maintenance -- and to contribute up to 20 per cent of its costs in kind. The Nepalese Ministry of Panchayat and Local Development channels administrative, technical and financial resources from its central office, through regional directorates, to the district level. The Ministry receives grants-in-aid and technical assistance from UNICEF and bilateral aid organizations. The donor agencies contribute up to 70 per cent of the total investment cost of the CWSS programme (Strauss, 1983).

Arrangements to provide assistance for pollution control projects and programmes will continue to be necessary in order to manage water quality effectively in developing countries. Of course, as discussed earlier, the costs of obtaining water quality objectives must be evaluated against the beneficial uses of the water.

4. Water quality management plans

Various incentive systems which can be applied to achieve curtailments in levels of discharges have been discussed. Institution building and technical and financial assistance necessary to implement a national water quality management programme have also been discussed. What remains is to consider what degree of curtailment of discharges is necessary for a given water quality objective and what technological control measures are preferred in order to achieve the desired results. This would be determined through the preparation of a water quality management plan.

An overview of the water quality management process is shown in Figure 2. As discussed earlier, the process begins with the identification of desired water uses in light of social, political and economic demands. Having determined the desired uses of a water body, the commensurate water quality objectives are then defined. At this point, a question is posed: does the actual quality of the water body meet the desired objectives? When assessing existing conditions, actual water quality would be determined through a programme of sample collection and testing and data analysis. When considering proposed actions, such as the siting of a new industrial facility, the actual water quality resulting from the change would be predicted through a mathematical model or other analytical means. In either case, if the objectives are not met, there is no cause for further action.

When existing conditions do not meet desired objectives, or when analysis of proposed actions indicates resultant violations, a need for intervention is established. A water quality management plan is a rational and orderly procedure for identifying and evaluating alternative control plans and arriving at a preferred alternative. A water quality management plan is comprised of several components: inventory of pollution sources; determination of cause/effect relationships; identification and assessment of alternative control plans; and ranking of alternatives.

The inventory of pollution sources should be as comprehensive as possible, including point and non-point sources, both natural and anthropogenic. Various sources of pollutants have been discussed earlier in this paper. Each source should be characterized by its mass rate and constituents. Responses of the water body to these inputs should then be determined by means of a model. Mathematical models are extremely useful tools in establishing these cause/effect relationships. Actual field data may be used to verify the model.

Alternative control plans should be identified and their induced responses tested by the model. A variety of controls are available to lessen the effects of pollution. Generally, controls fall into one of two groups: reduction of waste discharges, or increase of the assimilative capacity of the receiving waters. A number of alternative controls are summarized in Table 5. Of course plans may include a combination of controls.

Alternative control plans must be ranked on the basis of technical, economic and environmental feasibility. Plans that do not meet the desired water quality objective may be dropped from further consideration. Cost/benefit analyses should be performed. Standard engineering techniques will normally yield a good estimation of costs. It must be noted, however, that, while some benefits of water quality management are quantifiable, such as increases in fishery or agricultural production, others are not, such as improvements to public health or increased recreational opportunities. Thus, the use of a quantified cost/benefit ratio may not always be possible. Rankings will often have to be made on the basis of some qualitative consideration of intangible benefits. In a wider environmental context, rankings should also consider any secondary environmental impacts associated with the alternative controls.

The ranking of alternatives will result in the planners recommending one preferred alternative. Nonetheless, all feasible alternatives should be presented to the decision-makers. Social and political considerations may cause the modification of some alternatives and may affect the outcome of ranking. For this reason, planners should take these considerations into account as much as possible when formulating plans.

Once the decision-makers have selected the preferred control plan, the controls must be implemented by means of an incentive system. Incentive systems have been discussed earlier in this paper. A surveillance and enforcement programme should also be implemented to check and impose compliance with the plan.

5. An example

The Comprehensive Water Quality Management Project for Laguna de Bay, Philippines (Centeno and Adan, 1982) provides a good example. Laguna de Bay, the largest fresh-water lake in Southeast Asia (3.2×10^9 m^3), is used extensively for irrigation, commercial fishing and public water supply for neighbouring municipalities. The lake is also projected to be needed as a source of raw water supply for Metro Manila around the year 1990. The lake is fed by 21 tributaries draining an area of 3,820 km^2 including some 50 municipalities, as well as large agricultural areas and several industrial zones.

Studies of Laguna de Bay conducted in the early 1970s concluded that the lake had reached a critical stage of pollution as evidenced by eutrophic conditions, and that the eutrophication process was nitrogen-limited. Salinity intrusion from Manila Bay was determined to be a second concern. Construction of a hydraulic control structure was recommended to prevent intrusion while a comprehensive water quality management programme was recommended to control pollution from other sources. Subsequently, the Comprehensive Water Quality Management Project (WQM) was undertaken by the Laguna Lake Development Authority (LLDA) with financial assistance from several international development assistance organizations. LLDA first conducted in-depth studies to characterize the lake's existing water quality and to establish interim water quality standards and goals necessary to support the desired uses: irrigation, fishing and water supply. LLDA then made an inventory of pollution sources, determined current pollutant loadings and projected future loadings. It was determined that agricultural and domestic wastes were responsible for about 80 per cent of the annual nitrogen loading and were the cause of eutrophic conditions and frequent algal blooms. Moreover, nitrogen production was projected to double by the year 2000 if development were to continue unabated. Current loadings of organics, heavy metals and toxic wastes were largely contributed by industrial discharges and crop irrigation.

LLDA then developed control plans to achieve the desired water quality objectives. Recommendations included the adaptation of low-cost technologies for the treatment of point sources, and improved farming practices to control nitrogen losses in irrigation return water. Environmental impact assessment statements for the various control options were prepared to ensure their technical, economic and environmental feasibility. Cost-benefit analysis considered the cost of the hydraulic control structure and waste reduction measures versus the benefits accruing to agriculture, fisheries and water supply. A favourable ratio of 1.2/1 was calculated even without consideration of intangible benefits such as public health, aesthetics and recreation. Demonstration projects were implemented to prove feasibility and to generate popular support for the controls. Finally, LLDA looked into the adaptation of rules and regulations to provide a basis, i.e., an incentive system, to enable it to implement the controls and achieve the desired levels of reduction in pollutant discharges. A monitoring system of lake and tributary rivers water quality and waste-water characteristics was also established to assess the effectiveness of the pollution control measures implemented.

6. Public education

Government efforts to control pollution can be greatly enhanced by the support of its constituents. Education can foster a sense of both personal and communal commitment to environmental protection, particularly when such protection is correctly portrayed as a means towards improving quality of life. As discussed earlier, the degradation of water quality can have significant adverse effects on important sectors of a nation's economy.

Governments can consider several approaches to public education. The first is the introduction of environmental considerations into the formal science curricula of primary and secondary schools. Students should learn that the cause and effect relationships inherent in water quality management (and other environmental concerns, for that matter) are understandable, quantifiable, and, most importantly, controllable. Secondly, governments can use mass media techniques to generate support for environmental programmes. Certainly the advertising industry, and others, have demonstrated the effectiveness of using radio, television and news media to influence public opinion. Finally, governments can support the activities of environmental interest groups which have direct contact with their members, and which command some degree of media attention.

Conclusions

Just as water pollution problems reached near-crisis proportions in many industrialized nations during the past few decades, so too will developing countries be faced with similar water quality issues in the decades to come. Population growth and increased urbanization will result in greater quantities and concentrations of domestic wastes. Efforts to meet growing demands for goods and services, and for commodities such as food and energy, will be attended by increased generation of industrial and agricultural wastes. Indiscriminate disposal of such wastes may render water bodies unusable for vital purposes demanded by societies: agriculture, fishing, water supply and even navigation and industrial use in severe cases. The economic consequences of a development policy that does not adequately consider environmental protection may be dire.

Developing countries must begin today to address this important aspect of development. Governments need to assess what is now being done and what is lacking in the area of water quality management. Where deficiencies are noted, improvements must be made to the legal basis for pollution control activities, and to the institutions responsible for those activities. A comprehensive national water quality management programme should be planned and co-ordinated through the various participating institutions. The central activities in such a programme would be the development and implementation of an effective incentive system, backed by thorough surveillance and enforcement activities. Once a nation's institutional framework is sufficiently developed, rational water quality management plans, based on scientific analysis, must then be prepared and implemented. Towards this end, nations can begin to identify the desired uses of specific water bodies, adopt standards, and take inventory of waste discharges affecting or potentially affecting such uses.

Education of environmental specialists will be necessary for most developing countries to fulfil these functions independently. Educating the public and the polluting entities will greatly enhance their acceptance of a comprehensive water quality management programme and will contribute to its success.

The United Nations Department of Technical Co-operation for Development, and other United Nations agencies, stand ready to assist member nations in meeting this vital challenge. In the coming years, UNDTCD plans to assist policy makers through seminars designed to elucidate the technological aspects of pollution problems and to discuss in greater detail the ramifications of various policy options and institutional frameworks. UN/DTCD will also provide the services of various specialists, through its technical assistance programme, to aid governments in developing laws, institutions and national water quality management programmes. Technical assistance will also be made available for the scientific analysis of specific water quality problems and the formulation of solutions for pollution control.

Developing countries can and must take the necessary steps to protect their water resources for vital uses today and in generations to come. UN/DTCD is committed to helping governments meet this challenge.

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Table 1. Types of non-point pollutant sources

AGRICULTURE
- Cropland
- Pasture and Rangeland
- Irrigated Land
- Wood Land
- Feedlots

SILVICULTURE
- Growing Stock
- Logging
- Road Building

CONSTRUCTION
- Urban Development
- Highway Construction

MINING
- Surface
- Underground

TERRESTRIAL DISPOSAL
- Landfills
- Dumps

UTILITY MAINTENANCE
- Highways and Streets
- Deicing

URBAN RUN-OFF

PRECIPITATION

BACKGROUND SOURCES
- Native forests
- Prairie Land, etc.

Source: UNEP/WHO, 1983.

Table 2. Typical palm oil mill waste effluent

Parameter	Average Level of Pollutant	Range (Min. to Max)	Recommended Effluent Limitation
pH	3.7	3.5-4.5	6.0-9.0
BOD_5 (mg/l)	25,000	20,000-35,000	100
COD (mg/l)	45,000	30,000-60,000	1,000
NH_3N (mg/l)	30	20-60	...
Org. N (mg/l)	600	500-800	...
NO_3 (mg/l)	30	20-60	...
TDS (mg/l)	35,000	30,000-40,000	...
Suspended solids (mg/l)	25,000	20,000-30,000	500
Ash (mg/l)	4,500	4,000-5,000	...
Oil/Grease (mg/l)	7,000	5,000-10,000	...
Starch (mg/l)	2,000	...	...
Protein (mg/l)	3,000	...	...
Total Sugar (mg/l)	1,000	...	...
Flow (kg/kg FFB)	0.6	...	...
Empty Bunches (kg per kg FFB processed)	0.25	...	...

Source: World Bank, 1984.
Notes: TDS = total dissolved solids
FFB = Fresh fruit bunch
mg/l = milligram per litre

Table 3. Water uses and quality considerations

Water Uses	Important Quality Parameters
Power	Dissolved oxygen, pH
Flood protection	-
Irrigation	Dissolved solids, conductivity, sodium adsorption ratio
Potable water supply	Pathogenic organisms, toxic substances, turbidity, colour, hardness
Industrial water supply	Hardness, pH, dissolved oxygen
Navigation, transportation	Suspended solids, pH
Fishing (commercial and subsistence)	Dissolved oxygen, CO_2, pH, pesticides, heavy metals, pathogenic organisms
Recreation	Pathogenic organisms, pH
Nature conservation (wildlife and aesthetics)	Dissolved oxygen, pathogenic organisms, toxic substances
Waste disposal	Dissolved oxygen

Adapted from Pescod, 1974.

Table 4. Proposed water quality for tropical countries

Controlling water use	Stream standard: Quality parameter	Stream standard: Suggested level
Potable water supply	Most probable number of coliforms (MPN)	Effluent quality similar to the natural state of surface water.
	pH	6.5-8.5
	Dissolved oxygen	greater than 2 mg/l
	Arsenic	less than 0.05 mg/l
	Lead	less than 0.05 mg/l
	Chromium (hexavalent)	less than 0.05 mg/l
	Cyanide	less than 0.2 mg/l
	Phenolic substances	less than 0.002 mg/l
	Chlorides	less than 1,000 mg/l
	Total dissolved solids (TDS)	less than 4,000 mg/l
Irrigation	Total dissolved solids (TDS)	Not more than 400 mg/l where there is poor drainage, saline soil and inadequate water supply. Not more than 1000 mg/l where there is good drainage and proper irrigation management. Not more than 2000 mg/l where there are salt-resistent crops, good drainage, proper water management and low sodium adsorption ratio (SAR) of water.
	Sodium adsorption ratio (SAR)	Not more than 10 where there is poor drainage. Not more than 18 where there is good drainage.
	Boron	Not more than 1.25 mg/l where there are sensitive crops. Not more than 4 mg/l where there are tolerant crops.
Fishing	Dissolved oxygen	Greater than 2 mg/l. A level of 2 mg/l should not occur for more than 8 hr out of any 24 hr period.
	Pesticides	
	DDT	0.002 mg/l
	Endrin	0.004 mg/l
	B.H.C.	0.21 mg/l
	Methyl Parathion	0.10 mg/l
	Malathion	0.16 mg/l
	CO_2	12 mg/l
	pH	6.5-8.5
	NH_3	less than 1 mg/l

	Heavy metals	less than 1 mg/l
	Copper	less than 0.02 mg/l
	Arsenic	less than 1 mg/l
	Lead	less than 0.1 mg/l
	Selenium	less than 0.1 mg/l
	Cyanides	less than 0.012 mg/l
	Phenols	less than 0.02 mg/l
	Dissolved solids	less than 1,000 mg/l
	Detergents	less than 0.2 mg/l
Waste disposal	Dissolved oxygen	Greater than or equal to 0 mg/l

Source: Pescod, 1974.

Table 5. Summary of alternative controls to meet water quality objectives

Reduction of waste discharges	(i)	Change in production process
	(ii)	Change in raw material inputs
	(iii)	Change in product outputs
	(iv)	In-plant recirculation of water
Reduction of wastes after generation	(i)	Materials recovery and by-product production
	(ii)	Waste treatment
	(iii)	Effluent reuse
Increasing assimilative capacity of receiving waters	(i)	Addition of dilution water (flow augmentation)
	(ii)	Multiple outlets from reservoirs
	(iii)	Reservoir mixing
	(iv)	Reaeration of streams
	(v)	Salt water barriers
	(vi)	Spatial or temporal effluent redistribution
	(vii)	Dredging

Source: Adapted from Lohani, 1982.

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OPERATIONAL PERFORMANCE OF THE FmHA RURAL WATER SYSTEM PROGRAM

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ABSTRACT

Rural residents secure water from many private sources including wells, surface water diversions, and hauled water. Where present, public supply entities provide water from a central system referred to here as a "Rural Water System."

The Farmers Home Administration (FMHA) is authorized to provide grants and long-term low-interest loans ". . . for the installation, repair, improvement, or expansion of a rural water facility . . ." Grants and loans support 90 percent or more of rural water system capital costs. Interest-only payments in the first two or three years of a loan are followed by 38 or 37 years of level payment amortization. Funding of this program totaled to an estimated $10.22 Billion during Fiscal Years 1966 through 1983.

This study examined the physical, organizational, and operational characteristics of 108 FmHA-funded rural water systems with major construction in the 1970-77 period. Heterogeneity of physical and financial attributes characterized the subject systems. Many systems were well managed and operated, but more than half had moderate to severe financial problems. Most problems were attributable to inadequate revenues due to inappropriate water rate schedules or customers' use of alternative water sources. No system reported having a depreciation reserve and all were consuming their capital investments. Implicit subsidies were estimated as nearly equal to the principal amounts of the FmHA loans.

It was concluded that the FmHA program most nearly achieves these intended goals: "improving rural residents' access to adequate supplies of potable water", and "maintaining sanitary and healthful living conditions in rural areas". Goals of "urban-rural parity", and "making rural water-sewer services affordable" were less well attained, and may be unattainable.

OPERATIONAL PERFORMANCE OF THE FmHA RURAL WATER SYSTEM PROGRAM

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INTRODUCTION

During the nearly 50 years since its creation in 1935 as the Resettlement Administration, a federal agency mandated to stimulate the rehabilitation of rural areas, the Farmers Home Administration (FmHA) has provided low-cost long term credit to eligible public and private entities in the rural United States. As one of these entitlements, funding programs for rural water systems were first authorized in 1937. Since that time, periodic amendments of authorizing statutes have modified the scope and the borrower eligibility requirements of the rural water systems program, but the central intent of the program has remained the same: to make rural water supply systems more financially feasible, thus improving rural residents' access to adequate supplies of potable water.

The study reported on here examined physical, organizational, and operational characteristics of 108 FmHA funded rural water systems with major construction during the 1970-78 period. The management, operations, and finances of 26 sample systems were subjected to detailed analysis and implicit subsidies attributable to the low-cost FmHA loan program were estimated. As background to the study report, this paper provides introductory descriptions of the FmHA funding program, rural water systems, and rural water supply problems. Where appropriate, recognition is taken of assessment studies completed during the 1970s. The concluding discussion includes an assessment of the operational performance of the FmHA program.

THE FmHA RURAL WATER SYSTEM FUNDING PROGRAM

The Consolidated Farm and Rural Development Act, as amended, section 306, authorizes FmHA grants and low-interest loans ". . . for the installation, repair, improvement, or expansion of a rural water facility . . ." The facility (system) must serve primarily rural areas, and the service area cannot include any municipality of greater than 10,000 population. The applicant entity must show that the proposed system cannot be self-financed or financed at reasonable rates and terms by commercial lenders. Objectives of this FmHA program are described as providing basic human amenities, alleviation of health hazards, and promotion of orderly growth in rural areas. Priority is given to system proposals from public entities, to projects needed for compliance with the Safe Drinking Water Act, to projects that serve communities with less than 5,500 population, to projects that have promise of improving water system feasibility and efficiency, and to those that will serve low income populations.

Grant and loan funds can be used to pay the costs of distribution lines, wells, pumps, and related facilities. Under certain conditions, FmHA funding can be used to pay the costs of rural water system renovation or to purchase existing systems. The maximum loan repayment period is the least of: (a) 40 years, (b) the life of the water or waste disposal facilities, or (c) the length of the statuatory authorization of the borrowing entity.

Interest payments only are required during the initial two or three years. (In some states five or seven years of interest-only payments are allowed.) Loans are level-payment amortized over the repayment period with annual debt service payments. Prior to 1981, interest rates were set at 5 percent per annum. Present interest rates are 5 percent, midway between 5 percent and the market rate, or the market rate depending on income levels in the service area and the system's contribution to the alleviation of health or sanitation needs.

The grant program is intended to reduce annual per water user debt service costs for a FmHA-funded water or waste disposal facility (system). At the time the subject systems of this study were constructed the grant program was intended to reduce debt service costs to not more than: (a) 0.75 percent of median annual family income (MAFI) for the service area if that median was less than $6,000, (b) 1 percent of MAFI for the service area if that median was $6,000 to $10,000, (c) 1.25 percent of MAFI for the service area if that median was greater than $10,000. Grant funding could not exceed 75 percent of the eligible project development costs (50 percent prior to 1978). USGPO (1983), and FmHA (1979).

Long term objectives of the grant component of the FmHA program, identified through a review of the legislative history included: (1) maintaining sanitary and healthful living conditions in rural areas, (2) helping communities realize their full economic potential, (3) attaining urban-rural parity, and (4) making rural water and sewer services affordable. NDWP (1977).

Prior to amendments enacted by Congress in 1965 that added authorizations for waste disposal system loans, and water and waste disposal system grants, total funding commitments to the rural water system program were relatively modest. Subsequent to the 1965 amendments, appropriations increased, and have totaled to an estimated $10.22 Billion ($2.33 Billion in grants and $7.89 Billion in loans) during Fiscal Years 1966 through 1983. NDWP (1977), Gessaman & Janovec (1982), and USGPO (1983). Data for estimation of funding allocations to the water system program and to the waste disposal system program are not available. However, the FmHA program has been the dominant source of funding for the approximately 14,000 rural water systems now in place. Dempsey (1984). FmHA financed systems have been constructed throughout the country.

RURAL WATER SYSTEMS

In most instances, rural water systems provide potable water to small rural communities, farms, rural businesses, and nonfarm residences scattered across the countryside. The typical organizational unit is a special purpose unit of government (a water district), a cooperative, or a nonprofit corporation. Most are special purpose entities authorized to carry out only activities associated with the funding, construction, operations and maintenance of a rural water system. In this discussion, such entities and physical facilities are referred to as rural water systems without regard to the exact type of organizational unit or the makeup of the physical facilities.

The typical rural water system physical plant has some or all of these components: wells, pumps, underground pipelines, elevated or underground water storage tanks, treatment plants, and re-pressurization stations, plus ancillary facilities and equipment required for operation and maintenance of the system. In some instances, treated water is purchased from a municipality or other water-supplying entity, and the physical plant may include only a distribution system and necessary pumps and storage facilities. Rural water system customers often are geographically dispersed resulting in high capital costs per user and in aggregate. Design and construction costs are funded by combinations of user contributions, borrowed funds, and supplemental grants with FmHA being the dominant lender and grantor. The time period between the initial design of the system and its construction may be several years due to the vicissitudes of the loan and grant application processes.

RURAL WATER SUPPLY PROBLEMS

Rural residents secure water from many sources and use it for human consumption, livestock watering (a large volume use when livestock numbers are large), household use, yard and garden use, plus many incidental agricultural uses. Rural business firms use water for manufacturing or processing activities, cleaning and maintaining facilities and equipment, and for amenities or environmental aspects of their operations.

State and national data on water supply adequacy are not compiled. Estimates of the number of rural water users experiencing water supply problems are based on many approximations and projections. A US Soil Conservation Service study conducted in 1975 estimated that 36.4 million people (17 percent of the 1975 population) were served by noncentral water systems. (Noncentral systems are defined as those serving five or fewer households.) GAO (1980). A 1978 study of rural community water systems (public supply systems serving at least 15 year-around households or at least 25 year-around residents) identified more than 34,000 such systems. Viadya & Allee (1982). One year prior to the GAO report, the National Demonstration Water Project estimated that 25 to 30 million rural residents lack adequate water supplies. NDWP (1977). The exact number and location of rural households and businesses with inadequate water supplies remains undetermined though all reports indicate that their number is large and increasing.

THE RURAL WATER SYSTEMS STUDY

Rural water systems are complex and expensive mechanisms intended to provide adequate supplies of potable water to households, farms, and business firms in small rural municipalities and throughout the open countryside. This dispersion of customers (water users) ensures that rural water systems will provide notable challenges to persons involved in their design, construction and management. In most cases these challenges must be met under conditions of moderate to severe financial resource shortages. Unless water users can and will pay monthly charges sufficient to meet the full economic cost of water deliveries, financial shortfalls are inevitable. When they occur, the system manager and directors must make difficult decisions. The nature and consequences of these operating conditions are more comprehensively addressed in an earlier publication. Gessaman & Janovec (1982). Selected insights from a continuation of that study and from other studies are presented here.

Physical characteristics of subject systems

Examination of FmHA records in the states of Iowa, Missouri, Nebraska, North Dakota, and South Dakota resulted in the identification of 108 FmHA-funded systems with loan closings or near completion of major construction during the 1970 through 1977 period. State-to-state differences in water supply conditions and FmHA lending policies were evident. By state, principal characteristics of the subject systems were:

Iowa -- Five large systems serving an average of 718 open county customers per system. Four of the systems had municipal customers with the average number being 600. One system was under construction and was projected to serve 4,640 open country and 7,000 municipal customers. Completed systems had an average of 497 km. of pipeline, and delivered treated water secured from well fields or municipal suppliers. Ground water supplies in the service areas generally were limited and of poor quality.

Missouri -- Fifty-four systems varying in size from very small to moderately large. The average number of open country customers was 334. Sixteen of these systems served municipalities with an average of 431 municipal customers. Fifty-two systems supplied data indicating an average of 107 km. of pipeline per system. About one-half of the systems delivered untreated water from wells. The remainder delivered treated well water (most was purchased from municipalities). Ground water supplies in the service ~~areas often were limited~~ and of poor quality. Twelve additionl systems were under construction.

Nebraska -- Sixteen systems varying in size from very small to medium-sized. The average number of open coutry customers was 295. Six of these systems had municipal customers with an average number of 865 per system. The average pipeline distance for fifteen systems for which these data were available was 236 km. Most systems were in areas where ground water supplies were limited and/or of poor quality though some service areas are water-short only in drought years.

North Dakota -- Ten large systems serving an average of 924 open country customers and 1176 municipal customers per system. Completed systems averaged 822 km. of pipeline. All deliver treated well water and/or treated water purchased from municipalities. Most service areas had no recoverable ground water supplies. Four very large systems were under construction.

South Dakota -- Seven small systems serving an average of 48 open country customers per system. No municipal customers were reported. Systems averaged 37 km. of pipeline. All deliver treated well water and/or treated water purchased from municipalities. Most service areas had very limited poor quality ground water supplies. Eleven systems were under construction. All were quite large.

Within rather general policy guidelines, each FmHA state office administers the Water and Waste Disposal System program as it deems appropriate. Interactions between state office lending policies, the physical resource endowment, population density, and state enabling statutes were clearly evident. In North Dakota and Iowa, FmHA encouraged the construction of large systems and physical conditions of water availability and population distribution made large systems feasible. Early South Dakota FmHA program administration supported the construction or purchase by water users of small subdivision systems. Subsequent policies supported formation of the very large systems reported as being under construction. Missouri and Nebraska conditions and program administration supported wide variability in systems' size and complexity.

Capital costs and debt service

FmHA grants and loans provided an estimated 91 percent of the $231 million capital cost of the 108 rural water systems examined in this study. Cost estimates for completed systems for which it was possible to compile complete data sets indicated great variability in capital costs per customer and per kilometer of pipeline. Total capital cost per customer of the lowest cost system was $977 for a small Missouri system. Capital cost was highest for a South Dakota system at about $12,700 per customer. For calculation of capital costs per kilometer of pipeline (a commonly used measure of relative system cost and an indicator used in preliminary estimates of system construction costs), the subject systems were divided into two classes: (1) systems with their own water sources, and (2) systems that delivered only purchased water.

Analysis of costs for systems in these two categories disclosed only modest differences. Capital costs of systems with their own water sources ranged from $1,738 to $12,290 per km. with most in the range of $2,800 to $7,500 per km. Those that delivered only purchased water reported capital costs that ranged from $1,340 to $9,760 per km. with most in the range of $2,500 to $7,000 per km. In some instances, the costs of well(s) and treatment plant(s) appeared to be offset by the use of relatively more large diameter pipeline required because water was secured from a single source that was outside the system service area.

FmHA grants and water user contributions reduced per customer debt to less than $1,000 in some instances though no system received grants that approached the 50 percent of construction cost limit in effect during the period of construction of systems examined in this research. Most systems had per customer debt levels between $1,800 and $4,500 with the highest debt level being more than $7,000 per customer. When level-payment amortized over 37 years at 5 percent annual interest, these debt levels imply annual per customer debt service payments of $60, $108 to $269, and $419, respectively. Collection of median annual family income data (MAFI data) for system service areas was not within the scope of this project. However, comparisons were made to census of population data for household incomes in minor civil divisions that approximated a sample of the rural water system service areas. These comparisons indicated that in the higher cost systems, FmHA grants were not sufficiently large to reduce debt service payments to the maximum of 1.25 percent of MAFI specified in the program guidelines. This was consistent with findings of previous studies by federal executive agencies and the National Demonstration Water Project. CGUS (1977), GAO (1978), and NDWP (1977).

System revenues and reserves

More than one-half of the rural water systems examined had experienced moderate to severe revenue shortfalls. Water rate schedules differed greatly from system to system with only limited correspondence between a system's cost structure and the water sale rates (water charges). Factors that appeared to interact in determining water rates were: (1) the ability of customers to secure water from alternative sources, (2) the initial rate schedules adopted at the time water deliveries started (these usually were based on recommendations by the design engineer presented in the design study), and (3) the size of the system (small systems usually had lower water rate schedules). Many system managers indicated water rates should be increased, but customer aversion to increased charges for water prevented this from being done. Other systems -- especially large systems that served areas where no alternative water sources were present and those that had adopted relatively high initial rates -- were not experiencing severe revenue shortfalls. Thus, the situations of systems often seemed to be that of feast or famine. Either they were doing well, or they were doing very poorly. The consequences of revenue shortfalls were most evident in the ability of rural water systems to accumulate reserves, and in their ability to be fully current in their debt service payments.

FmHA standards call for accumulation of a reserve account with an amount equivalent to at least one annual debt service payment. This reserve is to be built up over the first 10 years of system life. It is intended to provide a "cushion" in event of revenue shortfalls and is intended to be a source of funds for replacement or repair of capital items. From a practical point of view, the FmHA reserve requirement is significantly less than good business management indicates should be in a reserve for capital replacement. At 5 percent interest with a 37-year period of debt amortization, the annual debt service payment is approximately 6 percent of the initial loan balance. Annual depreciation cost averages about 4 percent of fixed capital investment with some system components having expected useful lives of less than 15 years (pumps, motors, and possibly wells). If the loan is significantly less than the capital cost of the system due to debt reductions from user contributions or FmHA grants, a reserve that contained an amount equivalent to one year's debt service payment could be less than one year's normal depreciation of capital assets. As system components are used up, the expected value of capital items needing replacement inevitably will greatly exceed the amount that would be accumulated by a system in compliance with the FmHA reserve standard.

Most systems examined in this research had little or no reserves and gave little evidence of intending to be in compliance with FmHA standards. None was funding annual depreciation expense. None reported having a sinking fund for capital items replacement. Each was consuming its capital investment in a manner similar to that reported in 1977 by the General Accounting Office. GAO (1977). When asked, most persons involved in the operations and management of rural water systems indicate that financial resources needed to replace one or more major system components will be secured by further borrowing. This is feasible if FmHA or some other lender is willing to advance credit, but it will result in increased debt service payments. A limited analysis of debt service records for the subject systems created some doubt about the ability of systems to service more debt.

Debt service payment schedules and actual debt service payment records were compiled for a sample of 21 systems that provided complete data on their operations. Comparisons between these data series disclosed that, as of late 1978, the debt repayment achievements of these systems varied widely from system to system. When totals of interest and principal repayments that should have been made over the years since the system began operations and totals of interest and principal repayments that were actually made in the same periods were compared, some systems were more than current, and others were up to 50 percent in arrears. These comparisons indicated 14 of the respondent systems had aggregate debt service deficits of $766,907. The other seven systems had aggregate excess debt service payments (prepayments) of $64,932. By December 1983, FmHA reported that loans with payments in arrears in 1978 were considered to be fully current, though no data were made available that indicated when the back deficits were removed. One informant indicated that debt service balances unpaid after three years become a final balloon payment due at the loan maturity, and as long as the most recent three years payments are being made, the loan is considered fully current. At this point, documentation secured in the course of this research does not establish that the past deficits have been fully paid. Further work will be required.

Implicit subsidies

In addition to any direct subsidies that may occur through the apparently penalty-free deferrals of debt service payments described above, all FmHA loans accrue implicit subsidies attributable to below-market rates of interest. Any loan program that provides credit at interest rates below the market rates directly and continuously generates a subsidy for the borrower that continues throughout the repayment period. When the lender is an agency of a government that uses deficit financing, a reasonable measure of the opportunity cost of loanable funds for long term loans is the long term treasury bond rate. The quarterly average interest rate for 30-year US Treasury Bonds was selected as the most appropriate measure of opportunity cost for the estimation procedure reported here.

Implicit subsidies attributable to the FmHA loans received by rural water systems examined in this study were estimated for a sample of 20 loans selected by chance from the more than 150 loans extended by FmHA to the subject systems (many systems had two or more loans). These estimates were compiled through a three-step procedure: (1) Interest payments for the interest-only period and debt amortization payments for the repayment period were calculated for each loan using the FmHA 5 percent interest rate. (2) Comparable interest and debt amortization payments were calculated for each loan using the interest rate for long term Treasury bonds that was in effect at the time each loan was closed. (3) The totals of payments over the 40-year life of the loan under conditions (1) and (2) were calculated and the differences recorded. The loan principal amounts, the total payments under each interest rate, and the differences were separately totaled. The principal amounts of the 20 sample loans included in this procedure was $6,870,500. Total interest and principal payments over the 40-year terms of these loans at 5 percent interest was calculated as $16,266,061. Under comparable assumptions using the long term Treasury bond interest rates for the appropriate quarters, total interest and principal payments were calculated as $23,205,699. The $6,939,638 difference represents the implicit subsidy over the life of this group of loans. It slightly exceeds the principal amounts of the original loans, and provides a measure of the extent to which water users do not pay the full economic cost of the water delivered to them.

OPERATIONAL PERFORMANCE ASSESSMENT

The large number of rural water systems constructed throughout the United States under the FmHA Water and Waste Disposal System program gives clear evidence that the fundamental purpose of the program -- improving rural residents' access to adequate supplies of potable water -- is being achieved at a reasonable level, though the cost of that achievement may not be reasonable. Visible progess is being made toward the attainment of loan program objectives such as the providing of basic human amenities and, in areas of polluted water supplies, toward the alleviation of health hazards. Less can be said regarding its contributions to orderly growth.

The FmHA grant program objectives of attaining urban-rural parity and making water and sewer services affordable appear to be less well attained. Most urban residents have access to both sanitary water supplies and sewage disposal facilities. Many rural residents do not now have access to these services, nor will they gain access within the foreseeable future. There are fundamental physical, behavioral, and economic reasons for this condition. In some instances geologic conditions make construction of public water supply and sewage disposal systems overwhelmingly difficult and expensive. In some instances population densities are so low that even under conditions where geologic factors are not a barrier, the economic cost of public systems cannot be justified or repaid. And, in the arena of human behavior, some rural residents just do not want the amenities supported by the FmHA program.

In the more relevant situations where population densities justify public supply systems and geologic conditions do not prevent their construction, important questions remain regarding the performance of the FmHA program. Under most conditions, rural water supplies derived from public supply systems cannot be anything other than very expensive in comparison to the cost of comparable urban services. As long as the typical rural resident continues to believe that water is nearly free, ~~or should be no more~~ expensive that that supplied in municipalities, rural water systems will not be able to generate revenues that pay the total economic cost of supplying water through a rural water system. This research provides clear evidence that the customers of the rural water systems examined in the study are not paying the full economic cost of the water they receive. Even the most financially viable systems are not funding depreciation expense, and thus are consuming their capital. Some systems that have no reserves, do not fund depreciation costs, and have not been able to make their debt service payments have been collecting only a small propotion of the economic cost of the water they are delivering. In the short run they can live on depreciation and deferred maintenance and defaulted debt service payments. In the long run, it will take substantial infusions of public funds to keep these systems operating. The goals of parity and affordable cost may be not attainable.

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DETERMINATION OF FLOOD EVENTS FOR DESIGN PURPOSES

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ABSTRACT

During the last years several flood events with rather high return periods occured in south-west Germany, causing considerable damage also in rural areas. At the Institute for Hydrology and Water Resources Planning many investigations have been carried out to derive design values for channel improvement, detention basins, and other flood protection measures. According to the data situation, newly developed regionalized methods have been applied to determine the system function and the effective rainfall. The development possibilities of the different types of synthetic methods are also discussed in the present paper. Good regionalized models should be used even if rainfall-runoff data are available at the site.

In connexion with the determination of the height of the platform for nuclear power plants near the Upper Rhine hydrological and hydraulic computations have been carried out. The aim was to protect the site against a design flood with a frequency in the order of magnitude of 10^{-4} per year. In particular, statistical calculations of flood peaks and flood volumes have been performed, together with the determination of flood stages inside and outside the main channel, of flood waves caused by breaking of dikes, etc. Results of the calculation of flood attenuation caused by discharges exceeding the bankful discharge and the influence on flood frequency are shown.

Keywords: Flood control measures, design flood waves, synthetic (regionalized) rainfall- runoff models, effective rainfall, system function, unit hydrograph, nuclear power plants, design flood stage, flood attenuation, flood plain.

INTRODUCTION

In the Federal Republic of Germany efforts have been undertaken to improve protection against damage-causing floods in the last decades. Due to limited funds and the relatively small damage potential in rural regions, agricultural areas are only protected against minor floods (DVWK, 1983). This is advantageous in the downstream areas because the flood peaks are being reduced by flooding in the upper regions. But farmers asked for flood protection means in order to secure their income as well as to protect their villages. Such demands arose, in particular, because of a number of extreme flood events which occured during the course of the past few years in south-west Germany.

In order to prove the effects of alternative measures and to demonstrate the flood situation nowadays, it becomes necessary, to calculate flood events of different frequency. As it is not possible to calibrate rainfall-runoff models with the help of measured events, in many cases, methods have to be adopted which determine model parameters on account of easily derived catchment parameters (synthetic or regionalized procedures). During the course of the past few years models have been developed in Germany which make it possible to not only estimate system functions but also effective rainfall for small watersheds up to approximately 100 km^2 without on-site measurements (Verworn, Harms,1984; Lutz, 1985).

Nuclear power plants are often situated in thinly populated areas on big rivers. The height of their platform has to be determined according to German regulations in such a manner that floods with an exceedance probability of 10^{-4} per year cannot cause damage to the plant. Hydrological and hydraulic calculations for different sites in a wide river valley show that no inadmissible situations occur even in the case of extremely rare flood events if the level of the nuclear power plant equals the height of the dike crest. This is subject to the big cross-sectional area and the large flood plain.

APPLICATION OF SYNTHETIC RAINFALL-RUNOFF MODELS

Flood hydrographs may be determined by the transformation of effective rainfall into direct runoff with the application of system functions. The unit hydrograph has proved to be a suitable linear transformation function for small watersheds, especially by taking into consideration their dependence upon event-specific conditions (rainfall intensity, runoff coefficient, season). The effective rainfall often influences the peak value and naturally, the flow volume much more than the system function. The effective rainfall is that part of the areal precipitation which reaches the watercourse without passing the groundwater. At the Institute for Hydrology and Water Resources Planning of the University of Karlsruhe synthetic formulae have been developed for the estimation of effective rainfall and unit hydrographs by Lutz (1985). They have been based on data of 75 drainage basins (from 3 to 236 km^2).in the Federal Republic of Germany: forests from 0 to 100 %, and villages and towns from 0 to 85% of the total area. The development of the formulae of about 1000 rainfall-runoff events has been reported by Lutz (1985). This paper describes the application of the above-mentioned formulae in order to calculate design-flood hydrographs.

Effective rainfall

The effective rainfall of a non-urbanized catchment area is calculated, according to Lutz (1985) by

$$P_{eff} = (P - L_i) \psi_{max} + \frac{\psi_{max}}{a} (e^{-a (P - L_i)} - 1) \quad (1)$$

whereby

$$a = C1 \; e^{-C2/NW} \; e^{-2/QB} \quad (1a)$$

with P_{eff} effective rainfall in mm

P areal precipitation in mm

L_i initial loss (depending on soil type and land use) in mm

ψ_{max} maximum runoff coefficient, according to the U.S. SCS procedure (1972)

C1 regional factor (topography, soil type)

C2 land use and vegetation factor (2.0 - 4.6)

NW week number (31. week: NW = 1)

QB specific base flow in $l/s\ km^2$

For urbanized parts of the basin it may be assumed that the total rainfall of the sealed area - except 1 mm initial loss - reaches the watercourse. Presumably 30 % of the urbanized area are sealed.

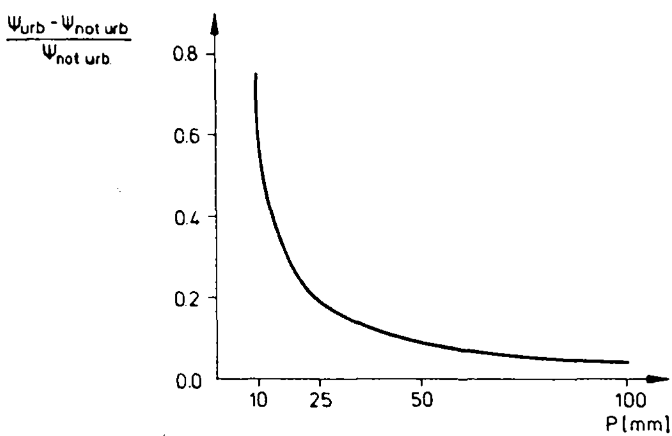
Different applications of the afore-mentioned formula have confirmed good fitting which is particularly based on the inclusion of the relevant parameters in a physically reasonable manner and the large number of used data. However, the maximum possible runoff coefficient ψ_{max} is obtained, in many cases, at extremely high rainfall depths.

Factor C1 should be calibrated if possible, by short-time measurements, or defined by transposition of rainfall-runoff results for neighbouring catchment areas. Table 1 compares the results of a 4-year-measuring period in a small catchment area at the spurs of the Black Forest with the synthetic results (generally recommended value for C1 = 0.02; calibrated: C1 = 0.034). A considerable improvement can be stated, especially in the region of high precipitation.

Figure 1 shows that a considerable relative increase of the runoff coefficient $\psi = P_{eff}/P$ for a rural catchment area may only be obtained for small rainfall depths. It was presumed that the urbanized part of the watershed increases from U = 0 to 10 % of the total area. This means that the volume of the direct runoff increases only essentially with frequent rainfall events of short duration. The afore-mentioned statement may only be applied for moderate future increase of urbanization in rural areas. Whether such effect leads to overloading the capacity of the watercourses depends on the design flood and the change of the system function.

Table 1 Example for comparison of measured and calculated effective rainfall

Date	Effective rainfall P_{eff} in mm measured	calculated C1 = 0.02	calculated C1 = 0.034
25.6.69	2.9	1.1	1.5
10.5.70	44.3	33.5	44.6
8.7.70	1.0	1.1	1.6
9.7.70	0.4	0.4	0.5
5.5.73	3.7	5.0	7.4
23.7.73	0.7	0.1	0.2



	10	25	50	100
$\Psi_{not\ urbanized}$ in %	3.8	12.6	24.5	40.7
$\Psi_{urbanized}$ (u = 10%)	6.4	15.0	26.7	42.5

Figure 1 Example for the relative increase of the runoff coefficient with the increase of urbanization from 0 to 10 % in dependence of the depth of precipitation

Unit hydrograph

The unit hydrograph $u(\Delta t,t)$ may be characterised by the time to peak T_p, by the peak value u_p, and by the shape of the dimensionless system function $u^*(t/T_p, u/u_p)$.

The following relations for Δt = 1 h are given by the investigations of Lutz (1985):

$$T_p = C \left\{ \frac{L \; L_c}{S^{1.5}} \right\}^{0.26} e^{0.004 \, F} \; e^{-0.016 \, U} \tag{2}$$

$$u_p = 1 \; / \; A^* \; T_p \tag{3}$$

with
- T_p time to peak in hours
- C factor (0.10 - 0.25) depending on the flow condition in the watercourse (Manning's k-value)
- L length of the watercourse (main channel) in km
- L_c length of the main channel from the basin outlet to the centre of the basin in km
- S slope of the main channel
- F forested area in percent of the total catchment area
- U urbanized area in percent of the total catchment area
- u_p peak value of the Δt-unit hydrograph in 1/h
- A^* area under the dimensionless unit hydrograph (= 1.6)

Formulae (2) and (3) contain only parameters of the catchment area and result in average unit hydrographs, the form of which may be derived from the shape of the dimensionless unit hydrograph.

According to Eqs. (2) and (3) an increase of urbanisation from 0 to 10 % - as presumed in Fig. 1 - results in a 15 % decrease of time to peak and an equally large increase of the peak value resp.

In general, no considerable change of flood flows in the region of the design discharge of rivers etc. may result from the change of the runoff coefficient and of the system function, caused by a limited increase of urbanisation. If a watercourse still shows its natural conditions and is thus able to only convey very small floods, considerably more frequent flooding is to be expected, in particular, due to the increase of the runoff coefficient than in a non-urbanised state of the catchment area. The location of towns and villages in the drainage basin is also of importance.

If the dependency of the system function of a specific rainfall-runoff event must be taken into consideration, the coaxial graphical relationship of Fig. 2 may be applied in order to determine the event-specific time to peak T_p'. Although, high intensity of rare summer rainfall events causes a short time to peak, this tendency is almost compensated by a corresponding increase of the runoff coefficient.

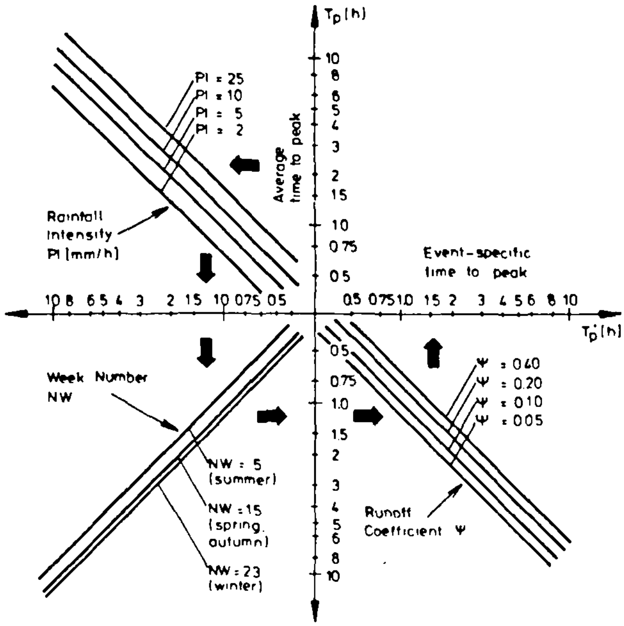


Fig. 2 Time to peak of the unit hydrograph (Δt = 1 h) in dependence of event-specific parameters

Application and development

Several applications of both synthetic procedures led to good results. The verification of extraordinary rainfall-runoff events of the past years and the comparison with the regional flood frequency were carried out.

It appears that all the information available has to be applied to the verification of results obtained by synthetic procedures. Each procedure contains basin-specific and event-specific parameters. These values cannot be estimated exactly. Moreover, good results are generally obtained for a certain region, but they may cause considerable deviations of the real result for a special catchment area.

"Worldwide" formulae to be applied to any region are not to be expected in the near future. Such formulae are supposed to be connecting all relevant parameters in a physically founded manner. Reasonable results also have to be obtained for extreme flood conditions, for which no data are available. The development of these formulae should be set about, although a large number of basin parameters are to be derived not only from easily available information (i.e. catchment area) and simple evaluations (i.e. L_c), but also by additional probably time-consuming evaluations (i.e. soil conditions). On one side, the application of further parameters may improve the accuracy strongly, and on the other side, part of that additional work may be left aside by calibration of coefficients (i.e. ψ_{max} in Eq. (1) and C in Eq. (2)), and the uncertainty with regard to the estimation of coefficients may be reduced.

The main advantage of such elaborate synthetic procedures consists in that the correct - physically founded - structure of this formulae also permits a certain extrapolation in such rainfall-runoff conditions, for which there are no measurements.

The accuracy of results having been obtained by
  a) analysed rainfall-runoff events only,
  b) regionalized rainfall-runoff models only, and
  c) (additionally) calibrated regionalized rainfall-runoff models
is qualitatively demonstrated in Fig.3.

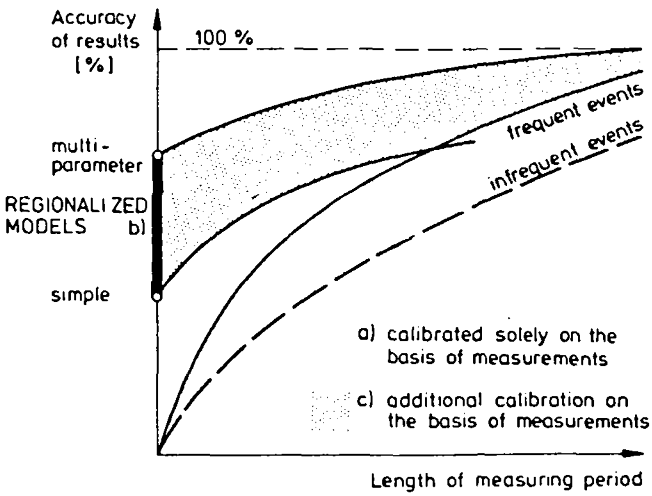


Fig. 3 Accuracy of results of different types of rainfall-runoff models in relation to length of measuring period

On account of the position of the curves towards one another regionalized models ought to be applied by all means, i.e. with the help of additional calibration of suitable parameters. That also counts for longer measuring periods at the river site of interest. The reliability of results, in particular with regard to rare events, may strongly be improved. Possible errors in the observation series are less effective. Outliers may thus be detected. Furthermore, synthetic models may pay regard to the effects of past and future anthropogenic changes in the catchment area.

DESIGN FLOOD FOR NUCLEAR POWER PLANTS

Along the Upper Rhine some nuclear power plants are planned or are in operation. In all cases, they are situated close to the dikes, which protect an area of about 8 km width, having been flooded by rare flood events earlier. The height of the platform for all nuclear power plants equals the height of the dike crest, decided by the responsible institutions.

It has been proved, that there is no danger for the atomic reactor even by extremely rare floods with a recurrence interval at a magnitude of 10 000 years. Flood waves due to dam breaks as well as wind set-up had been taken into consideration (Frank, 1951; Mitsuyasu, 1982).

The following demonstrates on how rare flood hydrographs attenuate in a wide river valley by retention.

The Rhine gauge Karlsruhe-Maxau is situated about 30 km downstream a reach consisting of river barrages designed for extremely high discharges. Flood frequency has been obtained by statistical analysis of an observation series of 113 years (upper curve in Fig. 4). Thus, the 10 000 year flood amounts to 7000 m^3/s. Bankful discharge of the Rhine amounts to about 5200 m^3/s for long distances, so that flooding of the above-mentioned lower areas of the Rhine valley is to be expected with higher discharges. The resulting reduction of the peak discharge was estimated with the help of hydraulic computations.

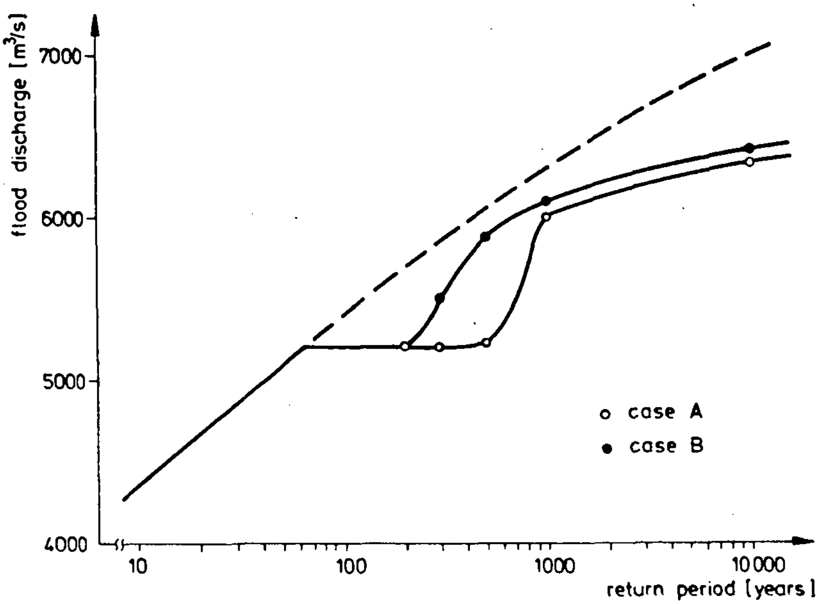


Fig. 4 Flood frequency for the gauging station Karlsruhe-Maxau, Upper Rhine (1871 - 1983)

When exceeding the bankful discharge two cases are differentiated:
(Case A) The dikes are overtopped without breaching and
(Case B) the dikes are partially eroded up to the ground resp.
The area left and right of the Rhine is divided into individual polders by dams (roads, railways). The water level of the polders had been presumed to be parallel to the ground, and has a slope of 3 $^{0}/oo$. The outflow from the polders takes place through outlets (roads, creeks) and/or by overtopping of the cross-dams. Calculation of flood attenuation was carried out for flood waves of different frequency according to the principle of reservoir routing. The wave shape was determined in such a manner that the observed relation between peak discharge and wave volume was followed.

The results of the calculations are also shown in Fig. 4. That figure shows that the polders in Case B get filled up faster than in Case A. The peak value of the input wave is less reduced in Case B than in Case A. An additional polder comes into operation at a recurrence interval of about 1000 years, so that the peak reduction again increases with extremely high return periods.

Computations for the site of a nuclear power plant, situated about 55 km downstream the last barrage, resulted in that the 10 000 year flood wave had been attenuated that much by then that the peak discharge is slightly larger there than the bankful discharge of about 5000 m^3/s. The inflows of the affluents in between are negligible.

CONCLUSIONS

Synthetic rainfall-runoff models make it possible to obtain design hydrographs for flood protection measures, in particular in rural areas, in which there none or hardly any measurements. In Karlsruhe, models for the computation of effective rainfall and unit hydrographs were developed. Further investigations are recommended in this paper in order to confirm system functions obtained by measurements for extreme conditions.

Flood routing calculations for extreme flood waves in a regulated river (Upper Rhine) have shown, that considerable peak reductions are to be expected with the exceedance of bankful discharge. Common distribution functions cannot be applied without modifications.

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LES TECHNIQUES NUCLEAIRES UTILISEES EN SEDIMENTOLOGIE DYNAMIQUE

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RESUME

Les techniques nucléaires sont de plus en plus souvent utilisées pour mettre en évidence et mesurer les mouvements sédimentaires. Elles complètent les méthodes conventionnelles en permettant des mesures directement in-situ. Ce sont maintenant, trente ans après leur premier emploi, des procédés éprouvés.

Les traceurs radioactifs, dont un large éventail est disponible, permettent, avec des activités limitées, d'estimer les transports sédimentaires (galets, sables, argiles) sous l'action des courants et/ou des houles. Les fines particules, souvent support physique et vecteur des polluants, sont marquées et suivies dans l'espace et le temps. L'interprétation de ces mesures lagrangiennes conduit à des modèles simples mais réalistes de la dispersion (trajectoire, dilution, décantation) des fines particules dans des conditions connues.

D'autres radionucléïdes à durée de vie courte, obtenus à partir de générateurs d'isotopes (couramment utilisés en médecine nucléaire) facilitent les mesures dans les canaux hydrauliques ou certains modèles physiques à fonds mobiles.

Les jauges nucléaires de turbidité, association d'une source radioactive émettrice et d'un détecteur de radioactivité, donnent la concentration des sédiments en suspension ou bien le profil vertical de concentration des sédiments fins déposés dans les ouvrages (retenues de barrage - bassins portuaires - chenaux). Ce sont des aides appréciés à la navigation (complément des sondeurs à ultra-sons), à la gestion des chantiers de dragage et à la programmation des chasses de barrage.

INTRODUCTION

L'industrie et le tourisme ont tendance à exploiter et à domestiquer avec brutalité le littoral, les estuaires et les fleuves : le recul des côtes sableuses est souvent de l'ordre de 1 à 10 m par an ; en France, l'extraction des granulats alluvionnaires, environ 2/3 des besoins, dépasse chaque année 200 Mt. "Erosion et défense des côtes" (1) et "Gestion régionale des sédiments" (2) sont bien des problèmes d'actualité.

L'augmentation des tirants d'eau des navires a entraîné le creusement des bassins portuaires et des chenaux de navigation. Le maintien des profondeurs dans ces zones, favorables à la sédimentation, nécessite de continuels et très coûteux travaux de dragage ainsi que la mesure précise des hauteurs d'eau qu'il n'est pas aisé d'apprécier lorsqu'il y a des dépôts de vase fluide (3) donnant par écho-sondage une image"floue" de l'interface liquide-solide.

Le comblement des retenues de barrage par des sédiments qu'il faut ensuite expulser par des effets de chasse pose des problèmes d'une ampleur reconnue (4) en particulier dans les pays semi-arides et tropicaux.

La surveillance réaliste de la qualité des eaux demanderait de prendre plus en considération la circulation et les temps de repos entre deux périodes de déplacement des sédiments très fins (argiles), support et vecteur de très nombreux polluants minéraux et organiques (5) d'origine industrielle, nucléaire, agricole et urbaine.

L'analyse de tous ces problèmes et leur résolution nécessitent une bonne connaissance de la dynamique des sédiments. Une appréciation convenable de la situation permettrait bien souvent d'importantes économies, d'éviter de graves erreurs, de sélectionner les sites à surveiller, de limiter le nombre d'échantillons au profit de prélèvements plus judicieux, etc...

La circulation des sédiments est due à l'action des éléments naturels (courants, houles, vents) mais aussi aux interventions humaines qui provoquent leur mise en suspension (rejets, dragages, extractions, vidanges des retenues de barrage). Selon l'importance des courants, suivant l'orientation et l'amplitude des houles, la géométrie ou la morphologie des fonds et la nature des matériaux (6), les érosions ou les atterrissements sédimentaires se développent de façon très différentes. Autant de paramètres variables dans l'espace et le temps s'ajoutant à des intéractions hydrauliques et sédimentologiques complexes, rendent la mise en équation et le calcul des quantités de matières solides transportées très délicats : les écarts d'un calcul théorique à un autre varient dans le rapport 1 à 100 (4) (7) notamment s'il y a simultanément alternance des courants de marée (8) et interférence des houles engendrées par des tempêtes épisodiques.

Il est donc souhaitable de faire appel à des procédés qui permettent des mesures directes. Les techniques nucléaires : traceurs radioactifs et capteurs radiométriques sont, dans le domaine de la sédimentologie dynamique, des outils particulièrement bien adaptés pour réaliser des mesures directement in-situ.

L'UTILISATION DES TRACEURS RADIOACTIFS

Leurs premiers emplois pour examiner les mouvements sédimentaires dans le milieu naturel remontent à 1954 (9, 10). Trente ans plus tard la méthode (11, 12, 13), éprouvée par près de 250 études, est de plus en plus souvent utilisée.

Dans la zone à étudier, qu'elle soit fluviale, estuarienne ou maritime, on introduit un sédiment (galet, sable, argile) marqué par un radionucléïde (tableau) émetteur de rayonnement gamma afin qu'il soit détectable dans l'eau et même recouvert par quelques dizaines de centimètres de sédiment naturel.

TABLEAU

LISTE DES PRINCIPAUX RADIOELEMENTS UTILISES EN SEDIMENTOLOGIE

Isotopes		^{113}In	^{198}Au	^{147}Nd	^{51}Cr	$^{170+181}Hf$	^{192}Ir	^{46}Sc	^{182}Ta	^{65}Zn	^{110}Ag
Période en jours		0,07 (100 mn)	2,7	11	27	40 et 70	74	84	111	245	253
Energie γ en MeV		0,3	0,41	spectre complexe E_{moy} = 0,26	9 % de γ 0,32	spectre complexe de 0,1 à 0,5 MeV	spectre complexe E_{moy} = 0,36	0,9 1,1	spectre complexe E_{moy} = 1	spectre complexe 1,1 MeV	spectre complexe
formes d'utilisation	marquage	x (bakélite) (vase)	x (vase) (sable)		x (vase)	x (vase)		x (vase)	x (galets)	x (vase)	x (galets)
	verre activable		x	x	x		x	x	x	x	x
Domaine d'emploi en sédimentologie		modèle hydraulique à fond mobile	mouvement de vase et de sable	mouvement de sable	mouvement de vase et de sable	mouvement de vase	mouvement de sable	mouvement de vase et de sable	mouvements de galets et sable	voir double marquage de vase	mouvement de galets

Les caractéristiques physiques : dimensions des grains, densité sont choisies identiques à celles des sédiments naturels.
Les galets sont marqués par introduction d'une petite source (souvent du tantale 182) dans un trou borgne ensuite scellé.
Les sables sont simulés par un verre broyé contenant l'un des radionucléïdes du tableau 1, généralement de l'iridium 192.
Les argiles sont marquées (14) par des processus physico-chimiques qui respectent leur comportement hydrodynamique et leurs conditions de floculation.

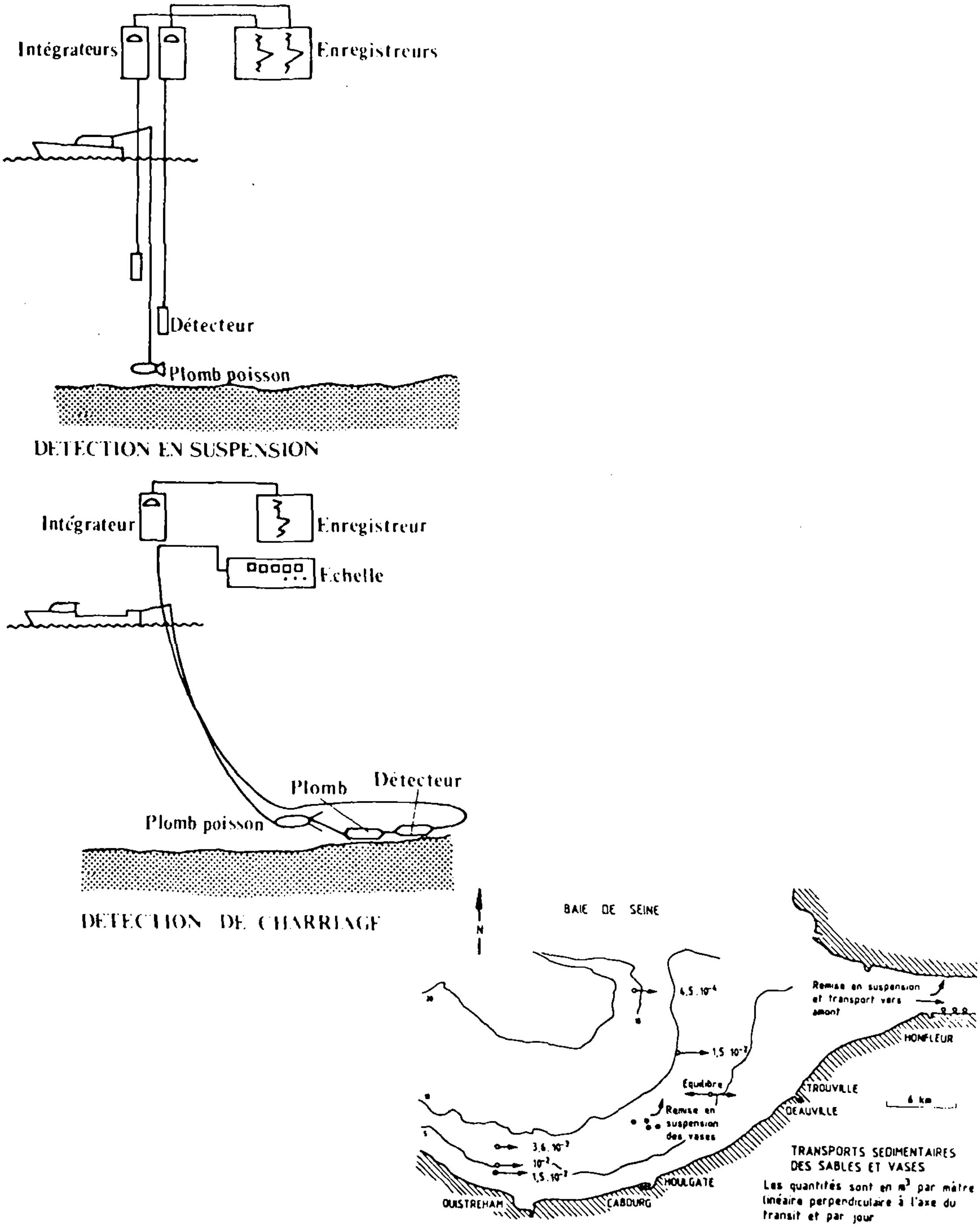


Figure 1 : Principe de la détection des traceurs radioactifs en sédimentologie et exemple d'étude d'un site

Les progrès constants des méthodes de détection et d'interprétation des mesures permettent assez souvent de n'utiliser par point d'immersion que le dixième de l'activité maximale autorisée par les conditions particulières d'emploi.

Au moyen de détecteurs immergés on mesure (figure 1) l'évolution des particules radioactives dans l'espace et le temps. L'examen de ces relevés permet de décrire qualitativement et quantitativement (12) les déplacements des traceurs en fonction des paramètres hydrauliques et météorologiques enregistrés par les moyens conventionnels.

Les résultats obtenus représentent la résultante de toutes les forces qui ont eu une action, *identifiée ou non, sur le sédiment marqué* entre deux prospections. Ainsi il est possible de prendre en compte les effets inconnus ou négligés dans les codes de calcul ou les modèles physiques.

Grâce aux traceurs radioactifs on obtient dans les conditions expérimentales de l'étude :

- la direction et la vitesse de déplacement des transports sédimentaires,
- les quantités de matériaux transportés,
- la proportion remise en suspension par les courants ou les houles,
- l'excursion et la dérive des particules en suspension,
- le taux de dilution des rejets en fonction du temps ou de la distance,
- la vitesse de sédimentation des particules en suspension.

L'introduction de radionucléïdes, identiques à ceux employés en médecine nucléaire, dans des maquettes et canaux hydrauliques permet d'étudier les mécanismes fondamentaux de la mise en mouvement et du déplacement des particules (15, 16).

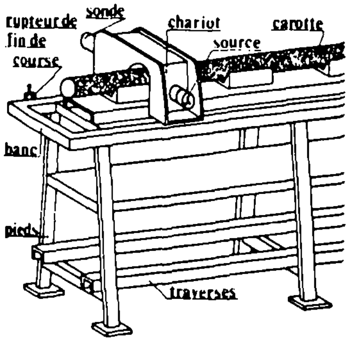
L'UTILISATION DES CAPTEURS RADIOMETRIQUES

Ces dispositifs, appelés aussi jauges nucléaires de turbidité ou de densité sont l'un des rares moyens non destructifs qui permettent de mesurer en continu soit des concentrations de particules en suspension, soit des profils de densité aussi bien au laboratoire sur des carottes sédimentaires que directement in situ dans des dépôts de vase en cours de consolidation.

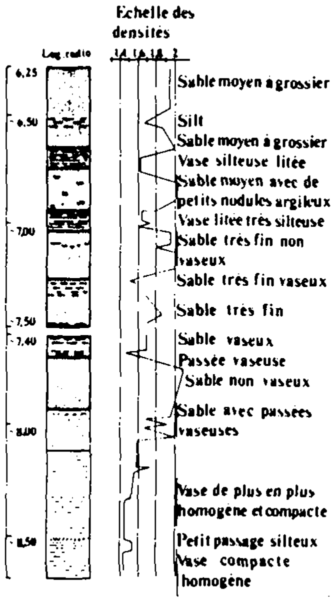
L'absorption et la diffusion des rayonnements électromagnétiques (gamma) émis par une source radioactive scellée sont fonction de la densité ou de la concentration des échantillons à condition que leurs géométries soient constantes et leurs compositions peu différentes de l'un à l'autre.

Les exemples donnés par la figure 2 illustrent le principe et l'emploi des jauges à transmission au laboratoire et sur le terrain (17).

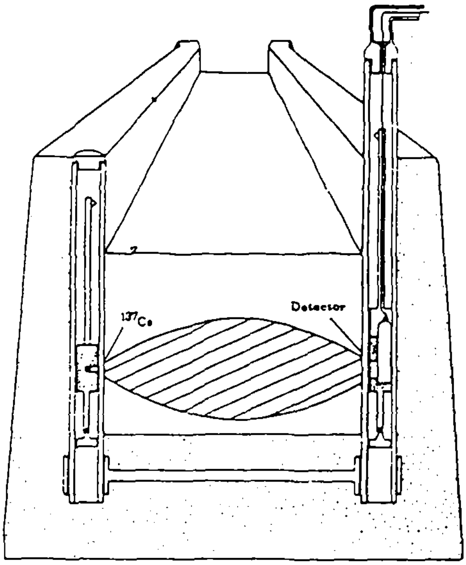
Les jauges à diffusion de photons, dont la forme géométrique et le poids facilient la pénétration verticale dans les sédiments fins en cours de consolidation, sont des aides à la navigation et aux travaux de dragage en complétant les informations obtenues par les sondages conventionnels.



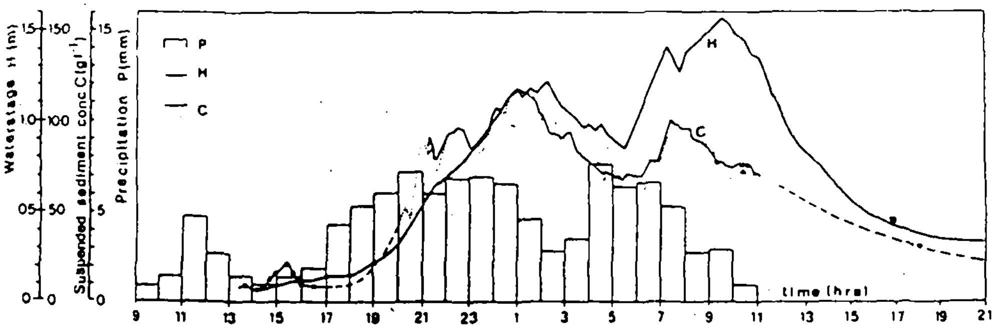
Vue en perspective du banc



Mesure de la densité de carottes
sédimentaires par absorption de rayonnement gamma
Réalisation CEA-SAR (France)



Station de jaugeage pour la
mesure de concentration de particules
en suspension (voir référence 17)



Mesures simultanées de l'histogramme des pluies et de la
turbidité à différentes profondeurs (d'après TAZIOLI - ref. 17)

FIGURE 2

La figure 3 résume l'intérêt et le principe des mesures effectuées avec un tel instrument :
La hauteur d'eau nécessaire à la navigation inclut l'espace (pied de pilote) entre la quille et le fond dur. Cependant dans les zones envasées, les navires peuvent se frayer un chemin, sans mettre en cause leur sécurité, dans la couche de vase non consolidée mais décelée par écho-sondeur comme si elle était dure (3).
Connaître le profil de concentration dans la couche près du fond est donc d'une grande importance non seulement pour la navigation mais aussi pour programmer les travaux de dragage : Quand et où draguer ? Jusqu'à qu'elle profondeur faut-il draguer ? Comment contrôler l'efficacité des opérations ?

CONCLUSION

Cinquante ans après sa découverte, la radioactivité artificielle est devenue un outil au service des chercheurs et des ingénieurs pour mesurer les transports sédimentaires. Elle complète les méthodes conventionnelles en étant souvent le seul moyen qui permette des mesures in situ.

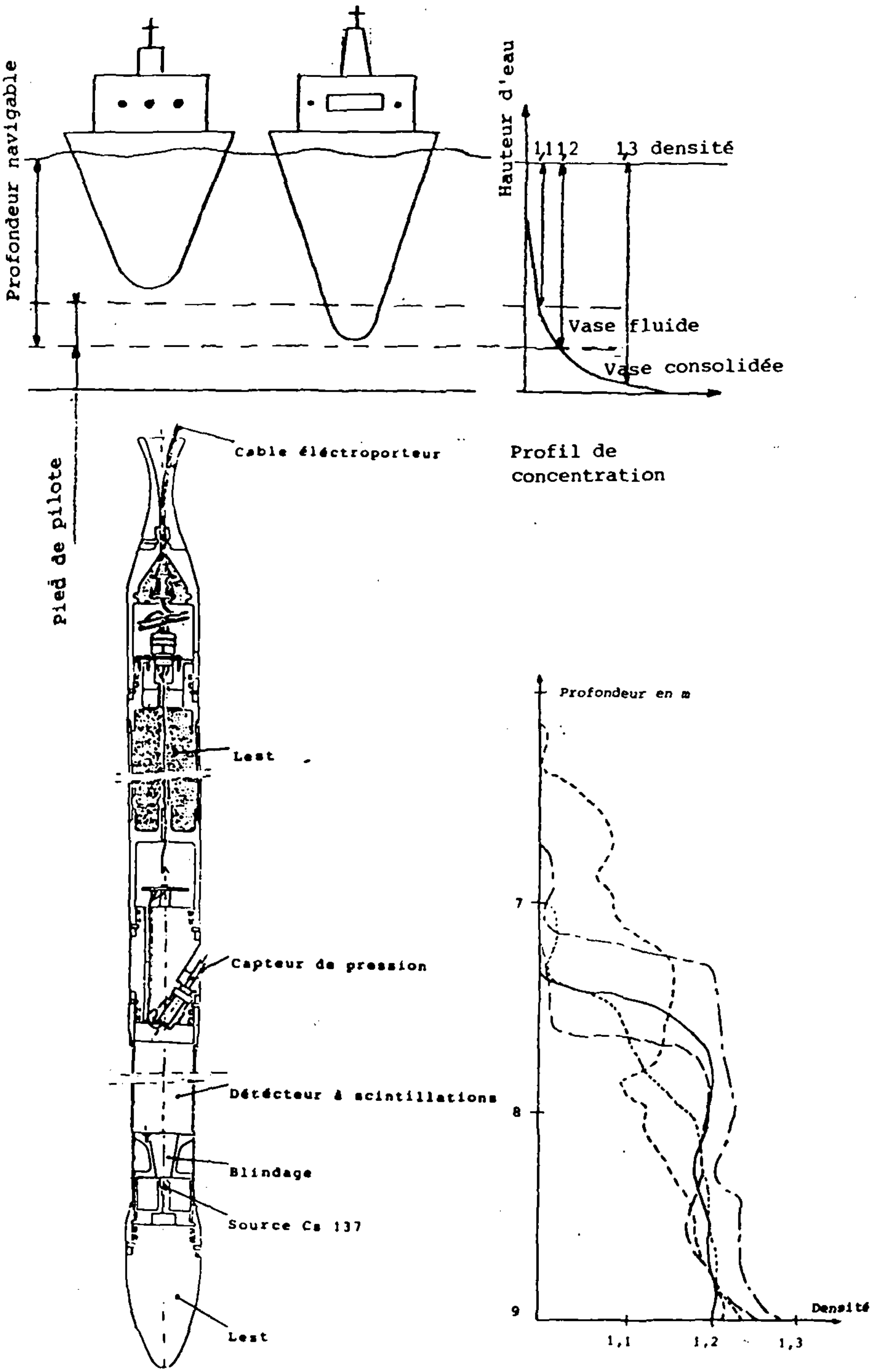


Figure 3 : Navigabilité dans les chenaux envasés
Exemple de profils verticaux de densité mesurés avec une jauge
à diffusion de rayonnement gamma
SAPRA JTD 3 réalisée par le
Commissariat à l'Energie Atomique, Service d'Applications des Radioéléments (France)

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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A SCHEME FOR WATER RESOURCES MONITORING IN RURAL AREAS

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ABSTRACT

One of the major problems in water resources management in rural areas is the lack of sufficient information, which demands a strategy for monitoring activities. In such cases, the location and sampling rates all depend upon the objectives of the planners.

The author proposes a new methodology called variance reduction analysis for optimal selection of sampling sites in random fields (e.g. groundwater table). This method is an extension of Kriging (i.e. a generalized Gauss-Markov estimator). The basis of variance reduction analysis is an information response function.

The analysis leads the planners to a loss function. This function incorporates specific objectives of the planners (e.g. minimizing losses due to information uncertainties) into an optimal sampling scheme. Two ranking functions are proposed. These functions represent information and economic gains due to more sampling. These two gain functions are utilized to identify the optimal location and number of new observation well sites.

Introduction

The planning and design of water resources systems requres a significant amount of information. Such information collection activities are usually very costly. For instance, in the United States a sample of groundwater from 100 feet down can easily cost over $1,000 if one counts drilling, engineering, and equipment costs that goes into the installation of an observation well [c.f. Goldstein, 1984]. These monitoring costs can be many times more expensive in rural areas of the third world. Thus, the planners are forced to search for optimal sampling strategies in order to increase the economic efficiency of their projects.

In addition, the above data, in particular the hydrological data, possess a significant level of uncertainty. These variations can arise partly from intrinsic or natural variability of the parameters. So some of the involved variables may be viewed as stochastic processes or random fields. For example the geohydrological variables such as the transmissivity, the storativity, and piezometric heads are of this type.

With such a perspective in mind, the data management of these variables can be studied in the framework of spatially distributed random variables. For such fields, the location and sampling rates are decisions that should be made by the planners. In many instances, the planners of water resources systems are faced with an insufficient amount of data. In the case of spatially distributed data, this insufficiency can be in two major forms. Firstly, there might be very little data available. Secondly, the measured values might be clustered together and therefore not provide sufficient information about the whole field. For example, the study of the water table data in the northwest of Kansas reveals that most measured values are clustered around major towns and farm communities [Rouhani, 1983]. Consequently, a significant portion of the whole region is not properly sampled. In such situations, the planners should design a data collection scheme in order to enhance their understanding of the field.

The questions that should be answered are:

1. Where are the best locations for further sampling, i.e., the optimal set?
2. What is the best size of the sample set, i.e., the optimal number of new measured sites?

These questions can also be expanded to include the best sampling rates (time dimension).

In order to answer the first question, there is a need to have a measure for the identification of the most uncertain sites. The planner needs a complete indicator of the relative influence of the added samples on the accuracy of the whole field in order to select the point with maximum gain (e.g., information gain). Furthermore, the planner should establish a ranking function in the von Neuman sense, which incorporates a preference structure for guiding decisions. Finally, the desired method should be applicable to cases where an explicit income or benefit function does not exist. An example is the collection of groundwater data on a regional basis.

Data Management Approaches

Prior to any sampling design, one should establish the objective of the study in order to deal with the question of data collection. There are two major approaches which are commonly used in water resources data management. In the first method, sampling procedures are designed based on maximization of the accuracy of the estimated field with budget constraints, or by minimizing the sampling cost subject to a criterion of minimal acceptable accuracy. Examples can be found in Fiering [1965], Matalas [1968], and Piemental [1978]. Such programs are suitable for regional studies, where the errors in data cannot be easily related to any monetary measure besides measurement costs. Thus the level of accuracy of variables has to be substituted for more common economic criteria such as economic benefits. These methods usually lack a meaningful interpretation of the optimal level of accuracy of the data. They will not tell the planner how much is gained by adding a new data point.

On the other hand, in the second approach the accuracy of parameters is interpreted in economic terms such as, Maddock [1973], and Ben-Zvi and Bachmat [1979]. This approach is applicable only to problems dealing with specific planning and management activities. Such programs yield more meaningful measures for optimal data management plans.

As mentioned in Rouhani [1983] the above methods each has its own deficiencies. The first approach puts heavy emphasis on the accuracy of results, but fails to interpret them in a meaningful manner. On the other hand, the project (farm) level models provide an economic interpretation for the accuracy levels, but appear to ignore the hydrological data. It can be inferred that the solution to the optimal data management lies in a proper marriage between the economic risks (or losses) and the accuracy of the hydrological parameters, such as the groundwater levels. The author proposes to utilize kriging as a method for the measurement of the accuracy of the estimated field in order to be related to a loss function.

The Kriging Method

The kriging estimation method is named after D.G. Krige who introduced the use of moving averages to avoid systematic overestimation of reservoirs in the field of mining. This idea was later generalized for dealing with spatial data by Matheron of Ecole Normale Superieure des Mines de Paris, Fontainbleau, France [c.f. David, 1977].

Kriging is basically a linear interpolation method for non-stationary random fields. In other words, given the values $Z(X_i)$, $i=1,\dots,N$ of a field $Z(X)$ at the data points X_i, $i=1,\dots,N$, kriging provides a technique to estimate the value of linear functionals of Z.

In point kriging one estimates the value of the random field at an arbitrary point X_o based on the given measured values in a linear form of:

$$\hat{Z}(X_o) = \sum_i \lambda_{io} Z(X_i) \tag{1}$$

where

$\hat{Z}(X_o)$ kriging estimates at X_o

$Z(X_i)$ measured values at X_i

λ_{io} kriging weight for $Z(X_i)$ to estimate $Z(X_o)$

The λ_{io} are defined by two criteria:

(1) Unbiasedness: $\langle\hat{Z}(X_o) - Z(X_o)\rangle = 0$, where $Z(X_o)$ is the underlying value of process at X_o, and

(2) Minimum square error or the "best" criterion which requires $\langle[\hat{Z}(X_o)$ $Z(X_o)]^2\rangle$ to be minimum. These conditions can be written in the following equivalent form:

$$\langle\hat{Z}(X_o) - Z(X_o)\rangle = 0$$

$$\mathrm{Var}[\hat{Z}(X_o) - Z(X_o)] = \text{minimum} \tag{2}$$

where $\mathrm{Var}[\hat{Z}(X_o) - Z(X_o)]$ is defined as the estimation (kriging) variance. These conditions are identical to the properties of the least square estimates which were illustrated by the Gauss-Markov theorem. Thus kriging can be called a generalized Gauss-Markov estimator.

The Variance Reduction Analysis

The kriging variance (2) can be utilized as a guideline for optimal sampling. In fact the area with the highest level of uncertainty can be picked for further data collection. However, such an approach ignores the over-all effect of a new measurement on the level of accuracy of the estimated field as a whole. The author proposes a new method to establish a measure for such an influence. This influence resembles a common "response" function. It tells the operator the level of improvement in the accuracy of $Z(X_j)$ due to a new measurement at X_i. This level of improvement is measured in terms of reductions in the kriging variances. Furthermore, this measure of variance reduction can be expanded to cover the whole field. Such an expansion enables the user to pick or rank the prospective locations for further data collections.

Rouhani [1983] shows that this response function which represents the amount of information gain, can be written as follows:

$$VR_{o*} = \frac{1}{V_*(N)} \left[K_{*o} - \sum_{i=1}^{N} \lambda_{i*} K_{io} - \sum_{p=1}^{\ell(k)} \mu_{p*} f_p (X_o)\right] \qquad (3)$$

where:

VR_{o*} = variance reduction at X_o due to a measurement at X_*

$V_*(N)$ = estimation variance at X_* prior to new measurement

K_{*o} = Generalized Covariance (GC) between X_* and X_o

λ_{i*} = optimal weight of $Z(X_i)$ in estimation of $Z(X_*)$ prior to the new measurement

μ_{p*} = Lagrange multiplier for the p^{th} monomial constraint in the Kriging system for the estimation of $Z(X_*)$ prior to the new measurement

N = number of existing data points prior to the new measurement

Now there is a need for a loss function to convert the above measure of information gain into an indicator for economic gains. For example in groundwater studies such a function can be defined in terms of over or under-estimation of $\hat{Z}$ (e.g., piezometric head estimates). Whenever $\hat{Z} - Z$ is positive (i.e., the estimated piezometric head is higher than the predicted one), the operators are faced with a penalty. These losses are in the form of higher capital costs of pumping. However, if the estimation results turn out to be underestimating the water table, the operators may in fact save some money in annual costs. It can be argued that these marginal losses and savings may not be equal. Considering all the above factors, one can define a loss function as follows:

$$L = \begin{cases} C_B(\hat{Z} - Z) & \text{if} \quad \hat{Z} - Z \leq 0 \\ C_L(\hat{Z} - Z) & \text{if} \quad \hat{Z} - Z \geq 0 \end{cases} \qquad (17)$$

where,

L = Loss function ($)

$\hat{Z}$ = Estimated piezometric head (ft)

Z = Actual piezometric head (ft)

C_B = Benefit per foot of under-estimation ($/ft)

C_L = Loss per foot of over-estimation (\$/ft)
C_B has to be less than C_L to guarantee the convexity of the loss function.

In order to evaluate the expected losses, one must make some assumptions about the statistical nature of the estimation errors. It seems reasonable to assume that estimation fluctuations (i.e., Z - Z) are normally distributed, with a zero mean and a variance equal to the so-called kriging variance which yields an expected loss as follows:

$$\langle L \rangle = \frac{C_L - C_B}{\sqrt{2\pi}}\sqrt{V} = cV^{1/2} \tag{5}$$

where

V = kriging variance $\langle [Z - Z]^2 \rangle$ (ft^2)

c = net loss coefficient $(C_L - C_B)/(2\pi)^{1/2}$ (\$/ft)

Equation (5) shows the expected losses at each estimated point as a function of the kriging variance at that site.
The following are defined as:

$$TOTV = \sum_j V_j$$

$$TOTSD = \sum_j V_j^{1/2} \tag{6}$$

where

TOTV = total sum of kriging variances

TOTSD = total sum of kriging standard deviations

Both TOTV and TOTSD can be redefined as weighted sums of kriging variances and standard deviations to reflect the relative importance of the different parts of the field. Furthermore, $TOTV_i$ and $TOTSD_i$ are defined as TOTV and TOTSD after the addition of the new data point at X_i. So TVR (i.e., total variance reduction) can be written as:

$$TVR_i = TOTV - TOTV_i = \sum_j VR_{ji} \tag{7}$$

TVR_i represents the total gain in accuracy or the information gain due to the measurement of X_i [e.g., Matalas, 1968].

Similarly TSDR (i.e., total standard deviation reduction) is defined as:

$$TSDR_i = TOTSD - TOTSD_i = \sum_j V_j^{1/2} - \sum_j (V_j - VR_{ji})^{1/2} \tag{8}$$

Now we can define total loss reduction due to measurement at X_i (TLR_i) as follows:

$$TLR_i = c(TSDR_i) \tag{9}$$

In other words, $TSDR_i$ reflects the economic gain due to a new measurement at X_i, while TLR_i represents the monetary value of added information. NEB_i (net expected benefit due to sampling at X_i) can be shown as:

$$NEB_i = cTSDR_i - MC_i \tag{10}$$

So the net expected benefit is directly proportional to $TSDR_i$.

Equation (10) can be utilized in two ways. First, all points that show negative NEB can be eliminated as potential data locations. Secondly the sites with positive NEB can be ranked as the best points for further sampling. As described earlier a set of weights may be assigned to different sites to reflect their relative importance. It makes this ranking procedure more flexible for different cases of data management.

Data Description

The available data are groundwater level observations made in January 1979 in Groundwater Management District No. 4 of Kansas, a rural area of nearly 5000 square miles in northwestern Kansas, including Sherman, Thomas, and Sheridan counties and parts of Cheyenne, Rawlins, Decatur, Graham, Logan, and Gove counties. The data set consists of 327 measurements made in water wells scattered at irregular locations within the district and outside but close to its boundaries. Average spacing between wells is about 3.6 miles. For study of the geohydrology of this region, readers are referred to McClain et al. [1975].

A subarea of about 2000 square miles is selected. This subregion lies between latitudes 38°48' and 39°48' North and longitudes 101° and 101°36' West. There are 84 measurement points in this area (see Figure 1). For actual data readers are referred to Rouhani [1983]. Northeastern and northwestern corners of this zone are rather densely measured, while central and southern parts of this subregion have relatively scattered data points. One notes this area has a diverse set of spatial characteristics in its data point distribution.

Summary of the Numerical Results

Based on the proposed algorithm discussed earlier, an optimal ranking of the prospective new measurement sites has been conducted. For the purpose of variance reduction analysis the field is divided into a 5 x 5 grid with $\Delta x = 8$ miles and $\Delta y = 16$ miles. The nodes are described as the set of potential sampling sites. At each round of kriging the point with maximum information gain is selected as the new added data point. The basis of this selection is the maximization of the added information. It was further assumed that the new measurements at each site corresponds closely to their estimated values. This assumption implies that the addition of the new data does not cause any change in the parameters of the following selected GC:

$$K(h) = 145.68\ \delta(h) + .89914\ h^3$$

Using the variance reduction analysis, the top 20 points have been ranked as the set of best locations for further measurement. Figure 1 illustrates the spatial distribution of the ranked sites. As expected, most of the added points are located in the lower section of the field which has few existing sampling sites. For example, 80% of the top 10 points are situated in the southern part of the region. In contrast, the central region of the upper section which was already densely measured, does not gain any new data point among the top 20 (see Figure 1).

Another look at Figure 1 shows that almost all nodes on the border lines are selected as sites for further sampling. 100% of the top 5 and 90% of the top 10 points are boundary nodes. Among the top 20, 15 points are located on the edge of the field. This is 94% of all possible boundary nodes. Meanwhile, the internal grid points get 5 sites which is only 56% of the total available internal nodes. It can be concluded that, given

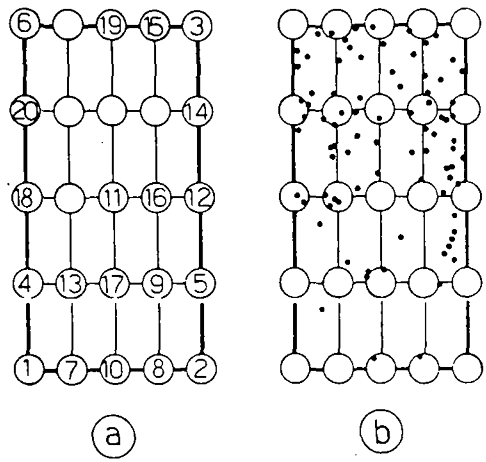

Figure 1: a. The set of selected points for further sampling, (Numbers in circles refer to thr rank of the point)
b. The set of existing data points.

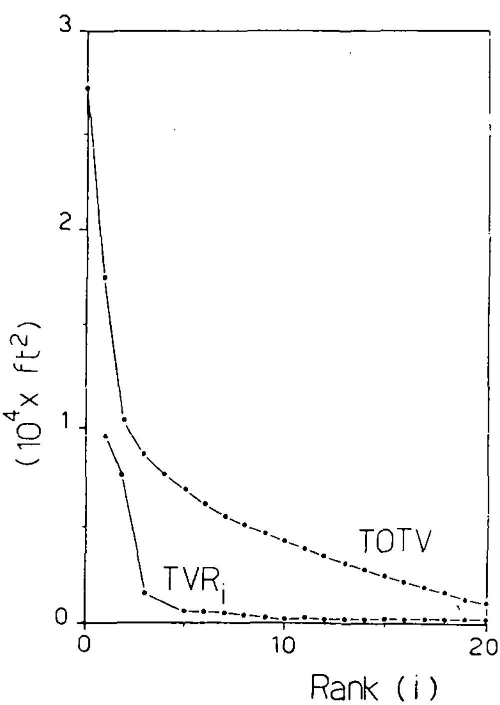

Figure 2: Total sum of variances and the corresponding marginal "information" gains due to additional sampling.

equal weights to each point, the boundary nodes are predominant choices for further measurements.

Figures 2 and 3 represent total variances and standard deviations at each round of kriging. They also show the corresponding marginal improvements in the accuracy of the estimated field due to the addition of each new point. As expected, both TOTV and TOTSD decrease as the number of new data points increase. These optimistic results are valid as long as the selected GC remains unchanged.

TVR and TSDR show the level of reduction in TOTV and TOTSD (i.e., improvement in the reliability of results) at each round of kriging. In initial rounds TVR and TSDR are quite high, but after few rounds they both approach almost asymptotic levels. This monotonic decrease in the values of TVR and TSDR resembles the concept of "diminishing rate of return" in economics. It indicates that as the number of new sites increases, the marginal improvement caused by additional measurements decreases. So there should be a finite optimal number of new measurements (N^*). In fact, one may add points until the NEB of the last added point becomes non-positive.

So the measurement cost/loss ratio of Equation (10) is the main indicator of the optimal size of selected sets. The above relationship is shown in Figure 4 (N^* as a function of MC_i/c). The shape of the graph indicates that N^* is extremely sensitive with respect to small values of MC_i/c (e.g., less than 20). However, as MC_i/c increases N^* becomes significantly less sensitive to the value of MC_i/c.

Summary and Conclusions

The author has developed an optimal data collection algorithm, i.e. the variance reduction analysis. This newly proposed method is based on an information response function along with a loss function. The two indicators of information and economic gains led to an optimal sampling scheme. Studying the pattern of selected points produced the following conclusions:

- Given equal weights to all nodes, border (extrapolated) nodes have a higher priority over internal (interpolated) points.
- Areas with low sampling density get a clear priority for further measurements.
- Marginal information and economic gains diminish to almost asymptotic values as the number of added points increases.
- As the measurement cost increases relative to net loss coefficient (i.e., MC_i/c goes up) the number of optimal new points (N^*) decreases.
- When MC_i/c is small the sensitivity of N^* with respect to the above ratio is far greater than the case of larger MC_i/c. So at low MC_i/c more accurate economic data is needed in order to produce equally robust estimates of N^*.

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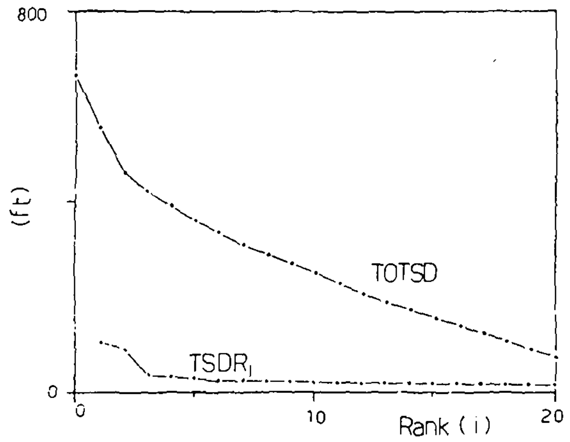

Figure 3: Total sum of standard deviations and the corresponding marginal "economic" gains due to additional sampling.

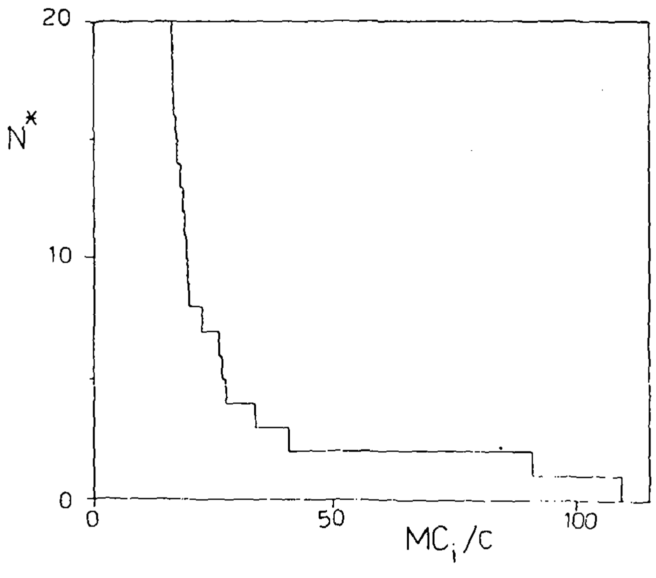

Figure 4: Optimal number of new data points vs. MC_i/c.

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RAPID SIZING OF RESERVOIRS IN DROUGHT AREAS WITH LIMITED RECORDS

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ABSTRACT

Development in some drought areas of Africa is dependent on available water. It is often difficult to justify large scale water schemes in such areas and smaller local dams are often the answer. As such dams are constructed on minor rivers, there is often insufficient record to permit accurate sizing of the dam. A method for simple estimation of the yield of such dams is described below. Regional records are extrapolated using limited parameters such as means and coefficient of variation and these also permit estimation of risk of failure. Alternative methods of estimation of dam yield are compared and a fit based on a curve fit using an extreme value type equation is described. The technique yields dimensionless storage-draft plots which also indicates the critical drought period and carry-over required. The same data can be used for simulation of the operation of the dam to permit account to be taken of evaporation and loss of capacity due to siltation.

Keywords: Critical period, drought, extreme value distribution, regional water resource development, storage-draft analysis.

INTRODUCTION

Many areas in Africa have undergone drought in the last few years. Although the level of development of grand water resource schemes has not been as high as it was decades ago, it is unlikely that internationally financed prestige dams on major rivers would contribute to the overall well-being of the areas which have been most afflicted by the droughts. The tribal people cannot be moved to larger schemes easily and in any case would require extensive training before optimal development could be achieved. It is only the smaller type of development which could alleviate the effects of drought to any significant extent. Dams constructed in small rivers, possibly using local materials and labour, would not carry the large volumes normally envisaged but would be more appropriate for supplying water to smaller scattered communities. Such dams would then in effect belong to the people and water could be led by them as required. The possibility of failure of such dams would probably be greater than those for major water supply schemes to urban development but with careful management and used in conjunction with ground water for emergencies, these dams would be far more economical than larger pumping projects.

In order to keep the costs of such development to a minimum, not only should construction techniques be developed for low cost construction dams, e.g. using soil fill and rock fill, but economical design methods must be developed. The planner will be particularly hampered by lack of river flow records in such areas and requires techniques for extrapolating what limited records he has to longer time durations as well as interpolating from river to river to assess the yield of ungauged streams. There are frequently very few gauges in such catchments, if any, and the engineer will be lucky if he has rainfall records, let alone stream flow records.

A technique is developed here for assessing the yield of dams which may have to carry over from year to year using a minimum number of parameters. Many attempts to fit synthetic mathematical functions to observe records have been made (Alexander, 1962; Fiering, 1967; Dincer, 1978). In general, the minimum number of parameters which can describe reasonably well the annual flow sequences of a river are the mean, the standard deviation or coefficient of variance and the skewness. In addition, there is often serial correlation to contend with and seasonal or monthly variations in the flow. The latter component is often the most significant in case of temperate climates whereas it is the annual flow which is of most concern in arid areas as carry-over is often required for more than one year. The coefficient of skewness can often be fixed for particular regions as well as the serial correlation coefficient which is a function of the climatic region. Seasonal or monthly variations often follow a similar pattern although the amplitude will be a function of the annual flow. There are thus two parameters remaining which must be obtained to assess the reservoir requirements. One is related to the mean annual flow at the site in question and this will usually be a function of the catchment area in square kilometers. These figures can be obtained by interpolating between other river gauges with reasonable accuracy. If necessary, the mean annual runoff could be refined by correlating with the annual rainfall and ground slope.

The main parameter affecting reservoir size in arid areas is however the variation in the river flow from year to year. The standard deviation of the annual flows is one measure of such variation and it is preferable to establish the coefficient of variation which is the dimensionless deviation obtained by dividing a standard deviation by the mean annual flows. The question as to whether to obtain the coefficient of variation of the low flows about the mean or the total record is difficult to answer however. It is usually the low flows which are of concern and owing to the inherent skewness in normal river flow records, it is the coefficient of variation if the flow is less than the mean which is of most relevance. This coefficient can however be frequently related to the coefficient of variation of the total record.

With a measure of mean and variation, it is possible to construct a model of the river flows using for example Chow's equation:

$$Q = \overline{Q} + st \tag{1}$$

where $\overline{Q}$ is the mean flow, s is the standard deviation and t is a random variate with a mean of zero and a standard deviation of one. The model can be extended to account for serial correlation and Brittan's model has an additional term as follows:

$$Q = \overline{Q} + r\,(Q_{i-1} - Q) + (1-r^2)^{\frac{1}{2}} st \tag{2}$$

The above models require a normal distribution of flow which is seldom the case. Attempts have been made to use the log of the flow which provides a skewness and ensures that flows are not negative. Again, such simplifications are made largely for convenience and not necessarily because that is the best fit. The next step would be to introduce the third parameter, namely, the skewness to attempt to model the annual flows more accurately. Where records are limited, it is difficult to estimate the skewness and the following technique avoids this by fitting an equation to the low flows. It is therefore assumed that the user is primarily interested in low flows and if a high flow should occur, it would break the drought so it would not be necessary to continue the analysis. It is also assumed that the relative frequency between droughts is reasonable so that reservoirs will have the opportunity to refill between the periods under consideration.

MATHEMATICAL MODEL

Chow's equation is rewritten in the following form for low flows:

$$Q = \overline{Q} - sK \tag{3}$$

where K is a function of the recurrence interval T. An approximation to K was found to be given by Gumbel's extreme value type I distribution:

$$K = (\sqrt{6}/\pi)\ (\gamma + \ell n\ \ell n\ T) \tag{4}$$

where γ = 0,57721 for large samples

$\overline{Q}$ = mean annual flow

Now the storage V required to meet a deficit in draft D over a 1-year drought commencing after a wet year in which the reservoir was filled is:

$$V = D - Q \tag{5}$$

$$= D - \overline{Q} + sK \tag{6}$$

If the drought extends over two or more (say N) years, the average flow may be estimated from the theory of samples if it could be assumed the flows are random. The expected mean of a sample is:

$$\overline{Q}_N = \overline{Q} \tag{7}$$

and the standard deviation of the mean is:

$$s_N = s/\sqrt{N} \tag{8}$$

Then the total flow over the N years will be:

$$NQ_N = N(\overline{Q} - s/\sqrt{N}\ K) \tag{9}$$

and the storage required:

$$V_N = N(D - \overline{Q} + s/\sqrt{N}\ K) \tag{10}$$

Using the equation the critical period N and corresponding maximum storage as a function of recurrence interval T can be selected by calculating the storage for alternative durations N.

For small drafts in comparison with the mean flow it will be found that the 1-year drought dictates the storage, and as the draft increases, larger critical periods will be evident.

SERIAL CORRELATION

For long droughts, the random sample indicated above may be unrepresentative. In fact, it will often underestimate the severity of a long drought because serial correlation is not accounted for. Climatic conditions often persist over a number of years and soil moisture deficit is likely to aggravate a drought in subsequent years.

Brittan's model (1961) accounts for serial correlation r. Replacing t by K as before for low flows,

$$Q = \bar{Q} + r(Q_{i-1}-\bar{Q})-(1-r^2)^{\frac{1}{2}}sK \qquad (11)$$

and for a sample,

$$NQ = N[\bar{Q}-r\,sK-(1-r^2)^{\frac{1}{2}}s/\sqrt{N}\,K] \qquad (12)$$

The storage required to meet a steady annual draft of D over N years of drought is thus:

$$V = N[D-\bar{Q} + \{r + (1-r^2)^{\frac{1}{2}}s/N^{\frac{1}{2}}\}sK] \qquad (13)$$

Storage may be plotted against draft with K as a parameter reservoir site provided $\bar{Q}$, s and r are known. Lines are plotted for alternative drought durations N and the critical period corresponding to the lowest draft thus selected.

The draft-storage relationships may be generalized by plotting $(D-\bar{Q})/s$ against V/s. Such graphs will apply regionally provided catchment size does not affect $s/\bar{Q}$ (the coefficient of variation) and serial correlation coefficient r.

APPLICATION

The method of obtaining $\bar{Q}$ and s for use in the above equations is to plot Q from an existing record against T using Gumbel's EV type I paper. Fit a straight line to the data over the region of interest and find the $\bar{Q}$ and s which apply to that line. These may not be the true mean and standard deviation which will be affected by the high flows as well. If the Gumbel distribution is applicable $\bar{Q}$ will be the 1,75 year value of Q on the line.

Fig. 1 is such a plot from which a value of $\bar{Q}$ of 1600 x $10^6 m^3$ per annum and a C.V. of 0.5 was reflected.

Fig. 2 is a storage-draft curve for $r = 0.15$ and T = 100 years recurrence interval of failure. The draft obtainable with a storage of say 2000 x $10^6 m^3$ would be obtained as follows:

Calculate $V/s = 2000 \times 10^6/(0,5 \times 1600 \times 10^6) = 2.5$

Then for Fig. 2, $(D-\bar{Q})/s = -0.5$

Hence D $= -0.5 \times 0.5 \times 1600 \times 10^6 + 1600 \times 10^6$

$= 1200 \times 10^6$ m³/annum

and the critical drought duration is 10 years.

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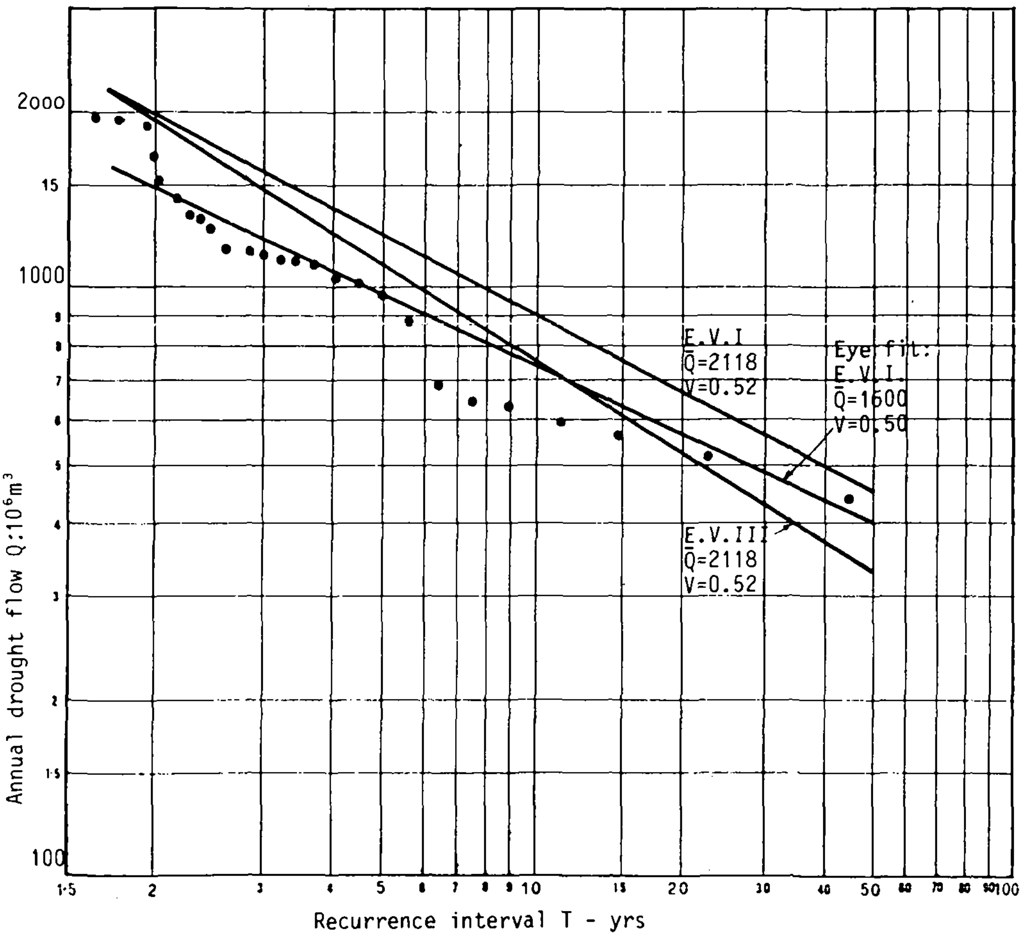

Fig. 1 Extreme value plot of annual river flows at Vaaldam

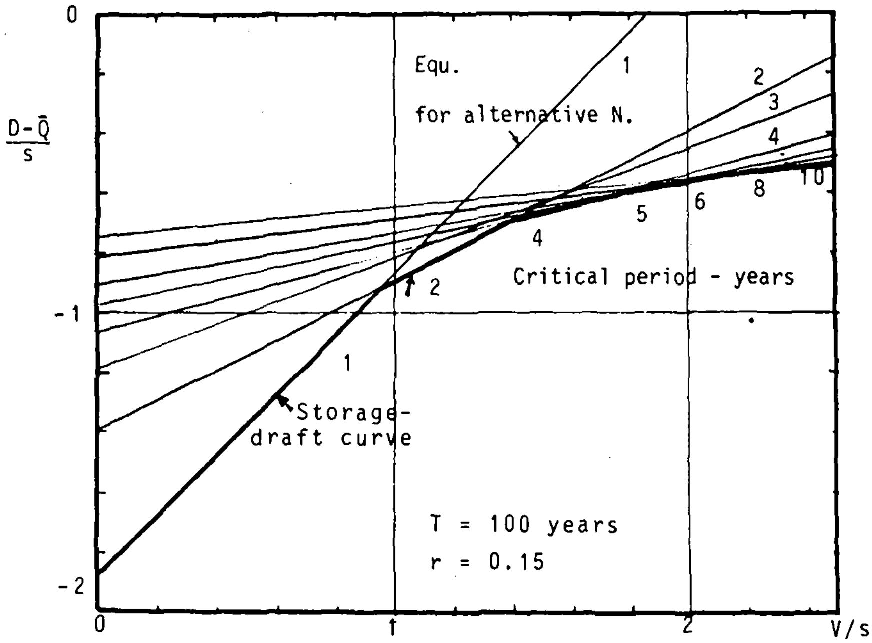

Fig. 2. Extreme-value Draft - Storage curve for lag-one serial correlation 0.15, recurrence interval 100 years

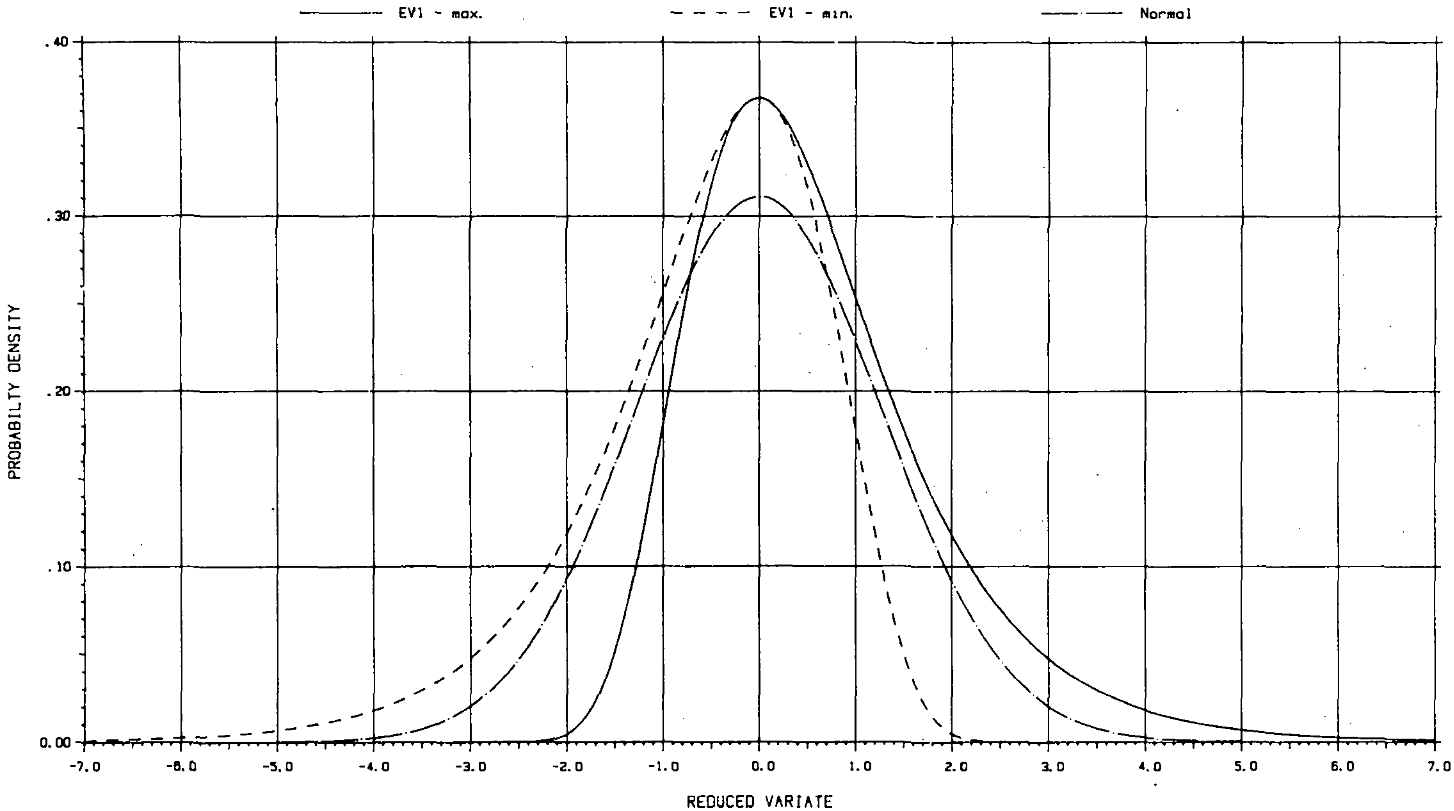

Fig. 3. COMPARISON BETWEEN EV1-MAXIMA, EV1-MINIMA AND NORMAL DISTRIBUTION

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FARM PONDS FOR RAINFED AGRICULTURE

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ABSTRACT

The State of Karnataka in South India is characterised by seasonal rainfall, with rainfall events interspersed with rainless intervals of a few days or even a month. This paper reports an experiment on the feasibility of farm ponds as a device to supply protective irrigation to dry land crops at critical stages of growth during such intervals. A pond was built in the middle of an experimental farm whose upper part served as the catchment and lower part as cropped land, where a crop of finger millet was raised. Five critical stages were identified, and water from the pond was applied during three of them coinciding with rainless intervals to target plots. An indigenous human-powered lifting device was used to pump the water. After harvest, it was found that grain yield in the target plots was higher by 90% than in the control plots and straw yield by 80%. A benefit-cost analysis has been made and the net benefit worked out.

INTRODUCTION

The State of Karnataka in south India, like the rest of the country, is under the monsoon influence, and the rainy season lasts from June to December, which is also the cropping season for drylands. The State (with a geographical area of about 190,500 sq.km) has a variety of climatic types, ranging from the superhumid coastal region in the west where the normal annual rainfall is more than 3500 mm to arid regions in the north and east with a normal rainfall of less than 600 mm. During the rainy season, the arid region has about 35 to 40 rainy days, compared to 125 or more in the coastal region. Since the cropping season lasts for 110 days or more, there is a high probability of extended rainless intervals occurring, ranging from a few days to even a month. If the crop is at certain critical stages of growth like flowering, grain formation etc when such drought intervals occur, there will be a significant reduction in the yield due to water stress.

Since a significant runoff occurs from several rainfall events, it is possible to store the runoff for later use. The present work was therefore taken up to examine the feasibility of using a farm pond to relieve the water stress. It is a pond excavated at a suitable point in cropped land, collecting runoff from the higher ground and storing it for use whenever required. The storage volume of the pond is necessarily small, since space occupied by the pond is lost to cultivation. The volume should therefore be just enough to keep the crop supplied with water at critical stages of growth (i.e., only protective irrigation is intended).

Table 1: Critical Stages of growth in Purna ragi

Stage	No. of days after sowing	Daily evapotranspiration at the stage, mm per day
Seedling	37	3.7
Tillering	51	3.7
Pre-ear emergence	59	3.4
Flowering	79	11.3
Grain Setting	95	5.2

An area of about 5700 m^2 was selected for the study in the Extension Centre of the Indian Institute of Science at the Village Ungra, 100 km from Bangalore. The village has a normal annual rainfall of around 700 mm. The land is terraced, and slopes towards the Keelara stream which is a few hundred metres away. The slope of the land averages 1 to 2%, and the contours are more or less parallel. In the middle of the area, the pond was dug in the form of a trapezoidal pit (Fig.1) extending along a contour for a length of 88.5 m. The top width of the pond is

4m, the bottom width 1m and the depth 1.5m. The storage capacity of the pond is thus about 332 m^3. The area above the pond (2788 m^2) serves as the catchment area, and that below the pond (2566 m^2) as the experimental field. The area is in the middle of an irrigated tract (practising irrigation by flooding), and in order to prevent excess irrigation water from other fields coming into the pond, drains were built at the periphery of the catchment area, which intercepted such water and conveyed it downstream past the area chosen for the study.

Downstream of the pond, an area of 1186 m^2 was selected for cropping. This area was subdivided into 8 plots as shown in Fig.1, each approximately 148 m^2 in area. The four plots (numbered 1 to 4) adjacent to the pond, called here the 'experimental plots', were to be provided with protective irrigation and the other four (numbered 5 to 8) were to be cultivated in the manner of any other unirrigated land (i.e., depending solely on rainfall). Ditches were dug as shown upto a depth of 50 cm. between the two sets of plots, so that soil moisture from the irrigated plots is not transferred to the root zone of the unirrigated plots.

THE CROP

The crop selected for the study was a strain of finger millets called 'Purna ragi' (Eleusina Coracana L),developed by the University of Agricultural Sciences, Bangalore. It is the staple food of a large part of the rural population of the State, and is first raised in a nursery and then transplanted into the field. Fig.2 shows its mean daily as well as cumulative consumptive water use (Reddy 1975). The total water requirement of the crop after transplantation is 51.5 cm. Large water demand occurs during the period from 40 to 55 days after transplantation (17% of the growing season), accounting for nearly 20 cm (39% of the total requirement). The critical stages of Purna Ragi (Duration 110 to 115 days from sowing) when water stress would reduce the yield, are identified in Table 1. Water requirement is more critical in the tillering and flowering stages than at other times.

THE WATER-LIFTING DEVICE

Field ponds can either be excavated(as in the present case) or partly excavated and partly formed by embankments (Krantz et al 1978).In the former case, runoff flows into the pond by gravity, but in the latter, it will have to be pumped into the pond since part of the pond storage is above the ground level. In either case, pumping is necessary to apply the water to the field.

Farm ponds should be small and therefore they benefit only small plots of land. The traditional lifting devices used locally, which are suitable for small plots, are therefore indicated for the job. For the present study, a yatha was selected as the pumping device. It was built out of a forked tree-trunk (the babul, a hard acacia species, was actually used) serving as a vertical post, on the fork of which a Casuarina pole was mounted as a lever with fulcrum at the fork, the shorter end of the lever on the water side of the vertical post and the longer end on the field side (Fig.3). From the shorter end, a hemispherical container was suspended from a bamboo pole as shown. The hinges at the fulcrum and the suspension point were simply made by tying ropes. A small platform was built on the bank of the pond near the foot of the vertical post for a man to stand on and operate the pump. The man would pull down on the bamboo pole till the container filled up with water and then release it. The weight of the longer arm of lever would lift the water container, which is guided by the operator to the head of the field channel situated near the vertical post. Repetitions of this operation were found to deliver an average discharge of about 10,000 litres/hour (more when the water level is high, less when low). The device would cost less than Rs.100 (US $ 9) to build, but its life would be short because of exposure of its wooden components to sun and rain. The life could be improved, at additional cost, by using stone slabs for the vertical post, treating the other wooden components against deterioration etc.

THE EXPERIMENT

The purna ragi seeds were first sown in a nursery at the rate of 74 gm per plot (5 kg/ha) on 27 September (271st day of the year, counting from 1 January). Farmyard manure at 400 kg per plot was applied on 3 October (277th day), Nitrogen at 0.74 kg and Potash at 0.55 kg per plot. The land was ploughed on 4 October, and the seedlings transplanted on 25 October (299th day of the year, i.e, on the 29th day from sowing). All the eight plots (1 to 8) had the same treatment in the above respects.

The plots numbered 1 to 4 were to receive irrigation at the critical stages identified in Table 1 in the event of insufficient rainfall occurring prior to those dates. Fig.4 shows the daily rainfall as recorded by the rain gauge installed at the Extension Centre (rainfall below 1 mm has been ignored). There were 61 rainy days, the bulk of the rainfall being received in the months of April, May, September, October and November, in confirmity with the normal pattern. The total rainfall for the year is 645.4 mm, which is about 8% below normal. The daily consumptive water use of the crop is shown superimposed on the rainfall diagram in Fig.4, on an elevated time axis. The five critical stages are marked on this diagram. It is seen that prior to the seedling and tillering stages, rainfall had occurred, and therefore no irrigation was provided. On the 59th day from sowing (329th day of the year), 6500 litres of water were applied to each of the four experimental plots (corresponding to a depth of irrigation of 43.8 mm), 10250 litres per plot on the 79th day from sowing (349th day of the year) corresponding to 69.1 mm depth of irrigation and finally 10000 litres per plot on the 96th day from sowing (366th day of the year) corresponding to 67.5 mm depth of irrigation.

The crop was harvested on 15th January (110 days after sowing) dried, grain and straw separated and weighed. The results are shown in Table 2, which gives the grain and fodder yields per hectare for the irrigated and unirrigated plots of this study as well as those obtained in Ungra village following traditional (rainfed) cultivation. It is seen that the ratio of grain yield to fodder in the experimental and control plots is in the region of 60 : 40, while that in the village is almost its reverse, being around 32 : 68.

Table 2 : Yield of Ragi(Grain+Fodder)

Plot	Yield in tonnes/ha		
	Grain	Fodder	Total
Experimental (with irrigation at critical stages)	3.49(59.4%)	2.39(40.6%)	5.88
Control (purely rainfed)	1.84(58.0%)	1.33(42.0%)	3.17
Village plots (Purely rainfed) (Ravindranath 1981)	1.23(32.3%)	2.57(67.7%)	3.80

The reason for this behaviour is not known, but is perhaps to be sought in differences in treatments like fertiliser application. Because of this, comparison with village yields may not be appropriate. However, comparison of the yields with protective irrigation to those without it under otherwise **similar conditions** shows that irrigation at critical stages can be rewarding : the grain yield goes up by 89.7% (from 1.84 to 3.49 tonnes/ha) and the fodder yield by 79.7% (from 1.33 to 2.39 tonnes/ha), the total dry matter yield increasing by 85.5%.

Cost-benefit ratio

A calculation can readily be made of the cost-benefit relationship for farm ponds.

The capital costs are:

(i) cost of excavation of the pond
(ii) cost of the water-lifting device (pump)

The recurring costs are:

(i) cost of labour in water-lifting
(ii) cost of maintenance of the pump
(iii) loss of revenue which would otherwise have accrued from the cultivation of the area occupied by the pond.

The benefit consists of the additional revenue by increased production in the land which gets protective irrigation.

Let C_e = Cost of excavation/m^3

V = Total volume of water required to irrigate 1 m^2 of land for the whole season

C_p = Value of crop grown per m^2 of unirrigated land

C'_p = Value of crop grown per m^2 of irrigated land

a_p = Area of land occupied by the pond per m^2 of irrigated land

C_1 = Cost of labour to lift 1 m^3 of water

P = Capital cost of pump

P_m = Cost of pump maintenance per year.

Then the total capital cost per m^2 of irrigated land is:

$$(C_e V + P)$$

If the rate of interest is i %, the recurring cost on the capital is

$$\frac{(C_e V + P)i}{100}$$

The total recurring cost per year (apart from normal inputs), assuming only one growing season per year, would therefore be:

$$C_p a_p + C_1 V + P_m + (C_e V + P) \frac{i}{100} \text{ per } m^2 \text{ of land.}$$

The net additional income to the farmer per m^2 of land which is given protective irrigation would be

$$(C'_p - C_p) - [C_p a_p + C_1 V + P_m + (C_e V + P) \frac{i}{100}]$$

Using current costs, the net benefit in numerical terms can be readily arrived at. From the actual operation at Ungra, the values for some of the variables in the above expression are as follows:

a_p = 0.1924 m^2/m^2 of irrigated land

V = 0.18 m^3/m^2 of irrigated land

C_e = Rs.4/m^3 ; P = Rs.0.04/m^2 (on the assumption that one yatha costing Rs.200 to build can irrigate 1/2 ha. of land).

Considering only the grain yield from Table 2, and taking the current price of Rs.1.60/kg of ragi, one obtains

C_p = Rs.0.294/m^2 and C'_p = Re 0.558/m^2

The water lifting is normally done by the farmer himself for small plots of the kind under consideration here. However, calculating at a notional rate of Rs.3/hour, during which time 10000 litres of water can be pumped by the yatha used, C_1 = Re $0.3/m^3$, Lastly, the maintenance cost of the pump is taken as P_m = Re $0.01/m^2$ of irrigated land (which amounts to assuming that the pump is replaced by a new one every four years), i is assumed to be 15%. Substituting these numerical values in the expression derived above, the net benefit is Re $0.0294/m^2$ of irrigated land. For 0.5 hectare, the net benefit would be Rs. 147/year. There are two points which should be mentioned here. The first is that the design treated above is not the optimum. It provides for a storage capacity equal to the total volume of water required for the season. This, however, is not necessary since rainfall and runoff would be occurring at several stages during the season, and a storage capacity arrived at by taking into account the withdrawal from the pond and replenishment during subsequent rainfall would involve less capital expenditure, and the land area taken up by the pond would also be smaller. The net benefit calculated by the foregoing procedure would then be correspondingly higher. The second point is that in order that the water stored may not be lost by evaporation and seepage, some treatment for the sides and bottom of the pond as well as a cover are indicated. Since the costs for these are going to affect the net benefit adversely, an optimisation has to be done to determine the extent of treatment and extra storage to compensate losses for maximum net benefit.

CONCLUSIONS

The work reported here shows that farm ponds for dryland agriculture are a worthwhile proposition. Although the pond occupies area which would be lost to cultivation, the net benefit obtained even in a near-normal rainfall year is substantial, and in serious drought years it could make an enormous difference to crop production in areas where seasonal rainfall conditions similar to the area in which the study was conducted prevail.

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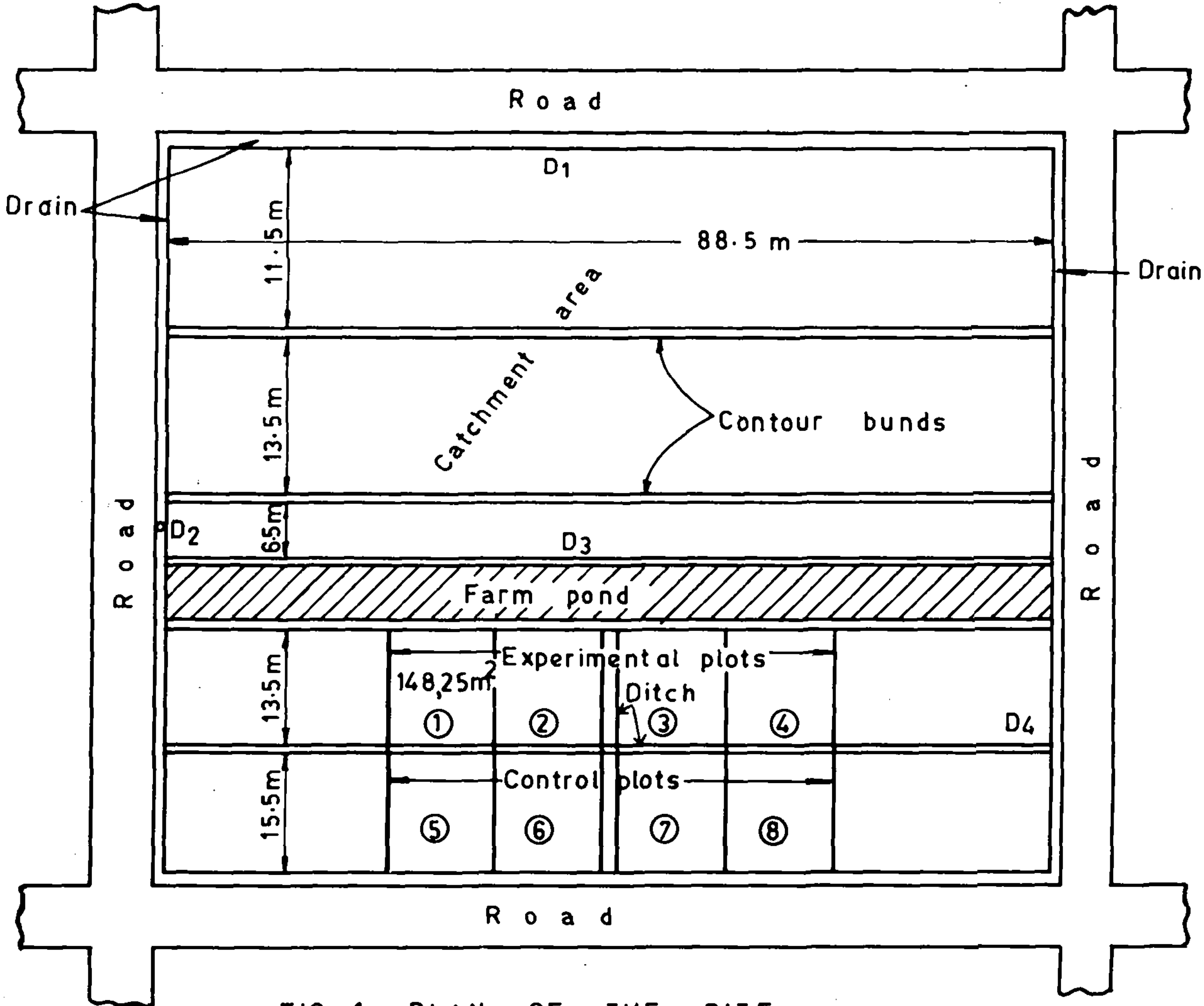

FIG. 1 - PLAN OF THE SITE

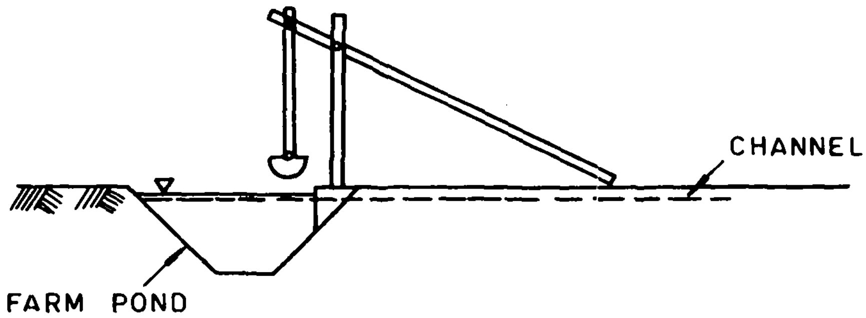

FIG.3. THE YATHA

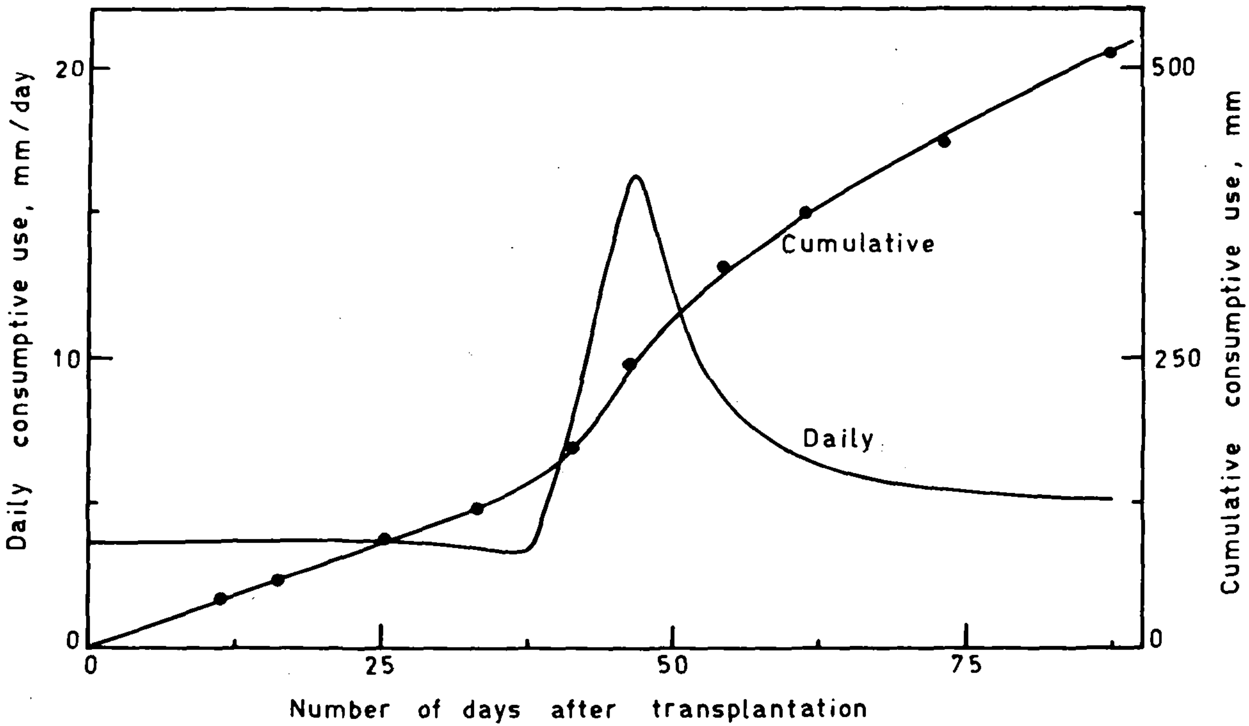

FIG. 2 -CONSUMPTIVE USE OF WATER BY PURNA RAGI

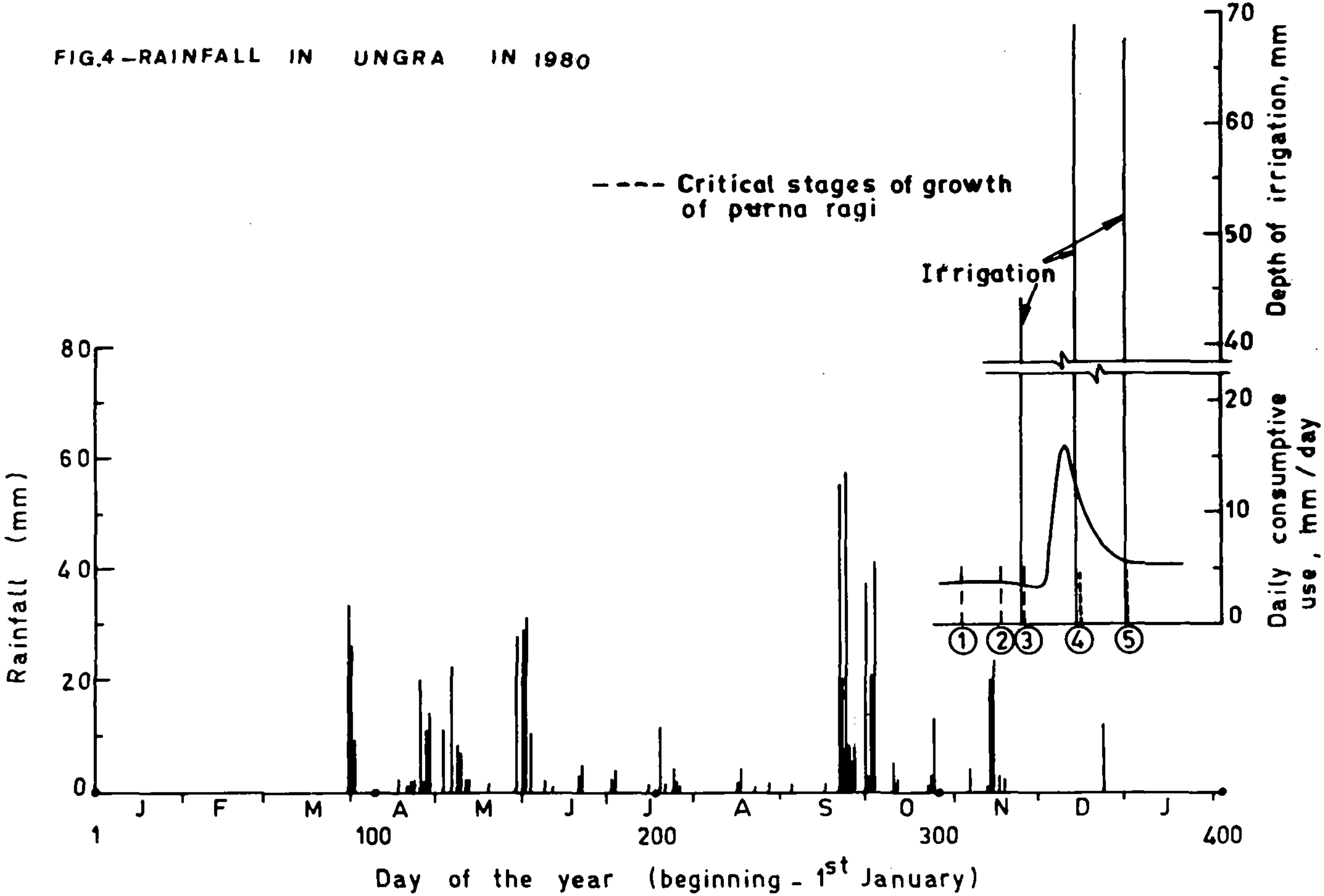

FIG.4 – RAINFALL IN UNGRA IN 1980

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ETUDE DES RESSOURCES EN EAU ET MICROPROCESSEUR

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RESUME

L'évolution foudroyante des circuits intégrés et tout spécialement des microprocesseurs a totalement révolutionné les techniques d'acquisition de mesures dans le domaine de l'étude des ressources en eau. En effet, la collecte de mesures en nombre suffisant a jusqu'ici bloqué pas mal de recherches à cause des budgets nécessaires.

Les microprocesseurs disponibles à ce jour sont d'une fiabilité extraordinaire. L'éventail de leur application est sans limite. Le but de la communication est de montrer quelques applications en la matière. Citons :

- Le développement de réseaux de mesure à transmission par réseau téléphonique commuté avec répondeur à microprocesseur. Ceux-ci permettent d'effectuer un prétraitement de l'information (par exemple : moyennes auto-étalonnage, etc..), d'analyser le phénomène localement et de déclencher automatiquement des interventions. Tout ceci se fait à des coûts fort bas en comparaison avec ceux nécessaires pour réaliser ces missions avec les techniques classiques.
- Le matériel d'acquisition à microprocesseur avec RAM et PROM CMOS. Ces appareils, à très faible consommation de l'ordre de 50 µA, ont une autonomie de plusieurs mois, une capacité de mémoire largement suffisante et une stabilité d'horloge meilleure que 2'/an.
- Les interfaces à microprocesseur disponibles avec des entrées et des sorties digitales et analogiques aisément programmables. Ce matériel permet, suivant une programmation déterminée, d'agir sur des systèmes parfois fort sophistiqués sans intervention humaine.

Tous ces développements existent et sont disponibles à ce jour pour ceux qui oeuvrent dans le domaine difficile de l'acquisition de mesures in situ pour les études des ressources en eau.

INTRODUCTION

L'évolution foudroyante des circuits intégrés et tout spécialement des microprocesseurs révolutionne aujourd'hui les techniques d'acquisition de mesure. Dans le domaine de l'étude des ressources en eau, ces nouvelles techniques nous ouvrent un nouvel avenir qui doit nous permettre d'envisager l'acquisition des mesures hydrologiques suivant d'autres formats. Ce sont ces nouveaux procédés qui nous permettront d'entamer des recherches à caractère plus exact.

DESCRIPTION DE QUELQUES NOUVEAUX MICROPROCESSEURS

Les microprocesseurs peuvent être définis comme étant la miniaturisation des processeurs.Rappelons que le processeur-computer pour les anglo-saxons est une machine qui permet essentiellement d'ordonner des processus de calcul. Les premiers résultent de la fin de la deuxième guerre mondiale. Le premier ordinateur aux Etats-Unis, construit en 1944, contenait 18.000 tubes. Son poids était de 30 tonnes. Il occupait une surface de 100 m^2. Il consommait 56 kWh. La probabilité moyenne de panne était de 7 minutes.

Depuis lors, une formidable évolution s'est produite. L'utilisation du transistor, dès le début des années 1960 et des circuits intégrés à partir des années 1970, conduit à ces fameux microprocesseurs d'aujourd'hui, (Wolfendale, 1972).
Leurs caractéristiques principales est la miniaturisation. Dans un circuit intégré de 36 x 20 mm à 28 broches de connexion, on peut trouver un CPU, 64 Bytes de RAM, 1100 Bytes de PROM, des interfaces de liaison.
Citons ci-après quelques caractéristiques de ces microprocesseurs.

Type	Consommation	T° de fct.	Technique
8035/39/48/49	135 mA, 1 µA Standby	0°, 70°	NMOS
80C35/39/48/49	1.5 mA Sleeping, 5 mA Operating	-40°, +85°	CMOS
6502	180 mA, 10 µA Standby	-40°, +85°	NMOS
65C02	8 mA Operating	-40°, +85°	CMOS
1802	10 µA Standby, 1 mA Operating	-55°, +125°	CMOS

CONCEPTION DE SYSTEMES DE MESURE

Les concepteurs de matériel d'acquisition de mesure dans le domaine qui nous intéresse - les ressources en eau - se trouvent confrontés à des options liées aux points suivants : la mesure, la transmission, l'environnement, l'autonomie, le coût.

Les options

La mesure

- Type : analogique ou digitale (niveau d'eau, météorologique).
- Multiple : plusieurs mesures sur un même site.
- Pas de temps : - pluie par pas de temps de 15', 1h, 24h
  - hauteur d'eau à l'heure ou moyenne horaire
- Précision : - hauteur d'eau 1 % sur fond d'échelle est souvent suffisant
  - pluviosité : 0,1 mm est universellement admis, mais entraîne des problèmes de taille mémoire

  - température : la norme de 0,1°C est souvent insuportable pour l'électronicien.
- Robustesse : le capteur hydrologique est placé dans des conditions très difficiles au milieu d'un cours d'eau pour la hauteur d'eau, au milieu d'un champ pour la pluie.

La transmission

La mesure enregistrée dans un support informatique local (mémoire RAM) est-elle :

- Télémesurée par radio, par câble
- Vidangée sur un support informatique localement.

L'environnement

- Conditions climatiques :
  - Température : -25°C à +55°C
  - Humidité : souvent proche de la saturation
  - Poussière : fine et abondante
  - Orage : parasites électroniques
- Conditions anthropiques :
  - Utilisation simple sur le terrain
  - Protection contre le vandalisme

L'autonomie

- Taille des mémoires :

  Vu l'évolution actuelle du coût des mémoires, le problème devient moins accru.

- Stabilité des horloges :

  Une dérive maximum de 3 à 5 minutes par an doit être exigée pour garantir les synchronisations.

- Energétique :

  Le raccordement au secteur doit être évité, parce qu'il coûte et qu'il est une source de pannes (absences de secteur et introduction de parasites lors d'orages).
  Vu les très faibles consommations énergétiques actuelles (quelques dizaines de micro-ampères), l'alimentation sur batteries rechargeables ou piles est impérative.

Le coût

C'est le point sur lequel le scientifique est le plus désarmé. Il doit abandonner son rôle d'hydrologue-électronicien pour devenir commercial. L'utilisateur quant à lui cherche à dépenser le moins possible. Il est cependant prêt à investir de gros budgets pour prévenir, sinon éviter, des catastrophes.
Malheureusement, trop souvent, le maître d'oeuvre croit qu'en investissant ces gros budgets il garantit la sécurité de fonctionnement de ces réseaux de mesures.

Les choix

Cette confrontation entre telle et telle option, les concepteurs la réalisent tous les jours.

Dans le domaine des ressources en eau, ce n'est que grâce à une association étroite de praticiens de l'hydrologie et de l'électronique que des développements efficaces peuvent se concrétiser. En effet, les écueils sont nombreux pour échouer dans un développement. Prenons trois exemples.

Hauteur d'eau

L'hydrologue cherche parfois une précision illusoire. Un limnigramme permet de lire une hauteur d'eau au mm près sur 6 m. Il faut donc exiger de l'électronicien une mesure analogique meilleure, soit 0,1 % du fond d'échelle, c'est-à-dire 2^{10} ou 10 bits de résolution. L'électronicien peut la réaliser. Mais c'est coûteux, alors que dans la majorité des cas, une résolution de 8 bits ou 0,25 % de fond d'échelle est largement suffisant.

Pluie

L'électronicien propose une taille mémoire de 2^8 = 256 pour l'enregistrement de la pluie. Or, cette taille est totalement insuffisante pour l'enregistrement de la pluie par heure, puisqu'à un basculement de 0,1 mm correspond une pluie de 25,6 mm/h. Il faut soit :
- porter la mémoire à 2^{12} = 4.096 (pratiquement à $2 \times 2^8 = 2^{16}$ = 65.536)
- adapter une procédure de stockage. Ainsi, dans la mémoire de 8 bits garder 7 bits pour enregistrer le nombre de basculement et garder le 8ème bits pour signaler que l'on stocke en 0,1 mm ou 1 mm.

La deuxième solution permet de doubler la durée d'enregistrement pour une même taille mémoire.

Analogique ou digitale

L'hydrologue ne se soucie guère de ce choix. L'électronicien sait que la mesure analogique présente toujours des difficultés telles que :
- une mesure analogique consomme (exemple : 4...20 mA)
- une précision dépassant les 8 bits est difficile et coûteuse
- une sensibilité extrême aux parasites.

Or, souvent avec un peu d'imagination, on peut développer des capteurs entièrement digitaux qui permettent d'éliminer les inconvénients précités (capteurs incrémentaux).

On pourrait citer de nombreux exemples. Développons ci-après deux exemples de configuration qui ont été obtenus en mettant autant que possible en application les constatations précédentes.

RESEAU DE TELEMESURE

Sur base de réalisations précédentes en télémesure par réseau téléphonique commuté (COPPENS d'EECKENBRUGGE et al., 1970; PERSOONS et al., 1974, PERSOONS et DE BRUYN, 1983; PERSOONS et LAURENT, 1981) des systèmes bidirectionnels à microprocesseur ont pû être développés.
Rappelons tout d'abord les bases de réseaux classiques monodirectionnels.

Réseau classique de télémesure monodirectionnel

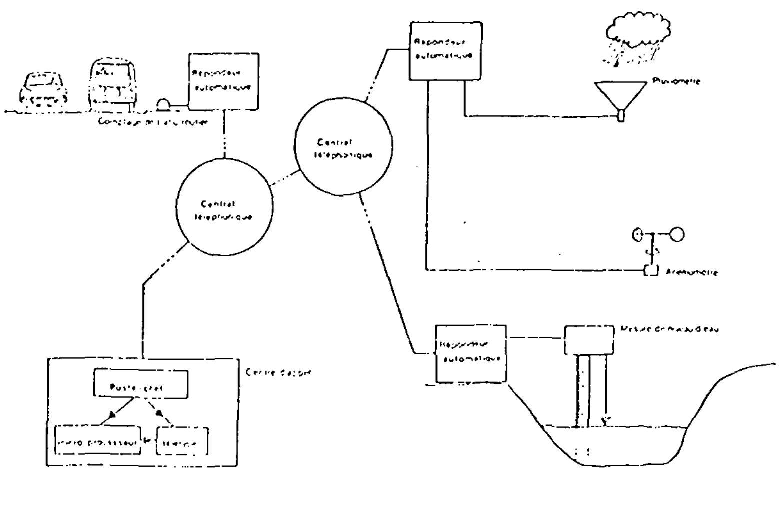
Principe général

Le système est basé sur une transmission de renseignements par ligne téléphonique commutée du réseau public.

A l'endroit de prise de renseignements est installé le répondeur automatique qui, sollicité par un appel téléphonique, se connecte à la ligne et émet une fréquence ou une série de fréquences fonction de la position du ou des capteurs auxquels il est relié. La figure ci-après représente un réseau de capteurs de niveaux d'eau, de pluie, de vitesse de vent, de trafic routier.

A l'autre bout de la ligne, passant par les différents centraux téléphoniques, est installé le poste-chef et son système d'enregistrement. Ce dernier possède une mémoire de plusieurs centaines de numéros de téléphone. Il appelle périodiquement chaque répondeur automatique à des intervalles préétablis variables de une fois l'heure à une fois la semaine.

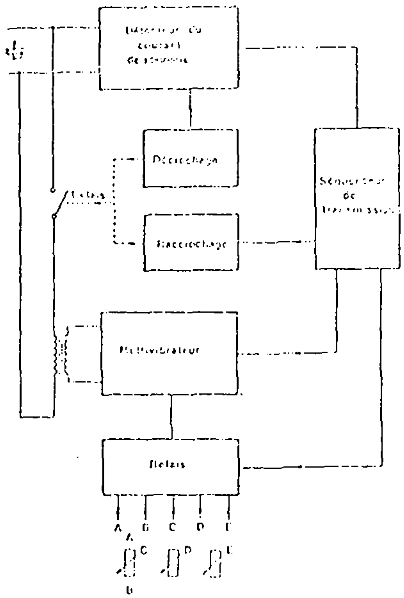
Les fréquences émises par le répondeur sont mesurées par le fréquencemètre à sortie digitale du poste-chef. Cette information est, soit enregistrée sur un support informatique, un ruban papier d'une télétype, soit transmise vers un système intelligent ordinateur ou micro-processeur, afin d'obtenir un traitement immédiat.



Réseau type

Répondeur automatique

Décrocheur-raccrocheur



Sur le terrain, à l'endroit de prise d'informations se trouvent les répondeurs automatiques reliés à la ligne téléphonique. Ces répondeurs comprennent :

Le décrocheur-raccrocheur (figure ci-contre) composé :
- d'un détecteur du courant de sonnerie qui commande le décrochage de l'appareil
- d'un séquenceur de transmission qui donne la séquence des mesures, commande la liaison vers les différents capteurs et le raccrochage en fin de transmission
- d'un multivibrateur qui transforme le signal mesuré en une fréquence. Ce multivibrateur est relié par des relais et des interfaces vers les différents capteurs.

Poste-chef

La version actuelle du poste-chef est construite autour d'un microprocesseur MOTOROLA 6800.
Les fonctions du poste sont :

- émission des numéros de téléphone
- établissement de la liaison avec les répondeurs
- mesure des fréquences reçues
- décodage et transformation des signaux en mesures directement utilisables
- transmission vers les périphériques ou vers des mémoires de masse.

Le poste-chef comporte :

- 1 à 4 interfaces pour lignes téléphoniques
- 1 horloge fournissant le jour, l'heure et la minute
- le CPU
- les mémoires PROM
- les mémoires RAM
- les interfaces asynchrones.

Le logiciel comporte de 8.000 à 12.000 instructions.

L'organisation des mémoires est la suivante :

- en PROM (Programmable Read Only Memory) :
  - le programme de base
  - les numéros de téléphone
  - les coefficients fixes
- en RAM (Random Memory) :
  - la gestion des appels
  - les coefficients variables (par exemple : les coefficients a et b de passage de la fréquence à la hauteur d'eau).

Les appels se font soit à la demande par instruction de type CALL 8.521, soit suivant une séquence prédéterminée. Un jeu d'instructions permet de programmer à la demande les séquences d'appel.

Réseau bidirectionnel

Il était assez logique de poursuivre les développements en passant du monodirectionnel au bidirectionnel, c'est-à-dire d'étendre l'application télémesure à :
- l'alarme : appel du terrain vers le poste central
- la télécommande : intervention du central vers le périphérique.

Conception générale

Dans le système monodirectionnel l'identification du poste appelé était aisé. En effet, chaque poste de terrain est identifié par son numéro de téléphone. Aucune erreur n'est possible. Le logiciel permet d'appliquer à chaque numéro d'appel un traitement adapté aux mesures effectuées "in situ" (Persoons, 1984).

Vu que dans le bidirectionnel le poste de terrain peut appeler le central il est indispensable de prévoir une identification. C'est la procédure "Hand-Chacking".

Configuration Hardware

La configuration est basée sur deux cartes de base et des cartes d'entrée et sortie enfichées sur un bus (PEP dans le cas particulier).
- une carte CPU sur base d'un microprocesseur M 6803 avec mémoires RAM et ROM et une horloge temps réel
- une carte interface de ligne reliant le système au réseau téléphonique commuté
- une ou plusieurs cartes entrée et/ou sortie type RS 232, digitale IN ou OUT, analogique IN ou OUT
- une alimentation sur secteur ou batterie en fonction de l'emplacement.

Configuration Software

La souplesse Hardware du système permet de développer un logiciel adaptable aux applications particulières. Il est composé d'un logiciel de base comprenant :
- la procédure de communication, y compris le Hand-Chacking
- l'acquisition des mesures "in situ" avec établissement de moyennes, détection de seuils, etc...
- les procédures d'envoi d'alarme
- les procédures de transmission de commandes.

Exemples d'application

- Alarme pluviométrique :

  A partir de la mesure d'un auget basculeur de pluviomètre, le logiciel peut détecter quand la pluviosité dépasse une courbe IDF (intensité, durée, fréquence) déterminée, et lancer une alarme vers le central.

- Télécommande de vannes :

  A partir de plusieurs mesures effectuées dans un bassin versant (pluviosité, débit, niveau de retenue), le centre d'appel peut envoyer une instruction de commande vers le répondeur-appeleur à microprocesseur. Cette commande peut modifier des ouvertures de vannes.

UNITE MICROMOS

Le mot micromos vient d'appareil à microprocesseur en technique MOS (Metal Oxide/Silicon transistor)
Cette technique, spécialement la CMOS (Complementary MOS) conduit à très faible consommation.

Unité de base

L'unité de base a été développée autour de microprocesseur type MOTOROLA ou COSMAG de RCA (Motorola, 1981; RCA, 1982).

A côté du microprocesseur est prévu :
- une horloge temps réel
- une mémoire PROM de 2 k byts (extensible)
- une mémoire RAM de 16 k byts (extensible)
- un périphérique d'acquisition digital ou analogique
- un clavier et affichage
- une sortie RS 232.

Les très faibles consommations de tels ensembles de l'ordre de 50 µA en état de veille conduisent à des autonomies locales de plusieurs mois, voir plusieurs années.

Avantages de l'enregistrement local en mémoire RAM (CMOS)

- RAM (Random Access Memory) : mémoire qu'il est possible de lire mais dont on peut également modifier le contenue sur place.
- CMOS : technologie de fabrication d'une mémoire entièrement statique qui consomme peu et uniquement lorsqu'elle est active.

Par rapport à un enregistrement graphique, ce type de matériel présente les avantages suivants :

- Il permet un traitement rapide et automatique par informatique. Il supprime le risque d'erreurs humaines au dépouillement.
- Au point de vue technique, l'entretien est pratiquement nul. Il n'y a pas de pièces mécaniques en mouvement.
- Ce matériel est moins encombrant, discret et donc moins sujet au vandalisme.
- Les réparations éventuelles se font rapidement et simplement par le remplacement de la carte défectueuse.
- L'autonomie de l'ensemble peut atteindre plusieurs années.
- La synchronisation des mesures est parfaite, la dérive de l'horloge est de moins de une minute par mois.
- Un même micromos peut recevoir les informations de plusieurs capteurs.

Exemples d'application

- Mesure de la pluie :

  Le développement actuel du MICROMOS permet d'enregistrer le nombre de basculement de l'auget par pas de temps de 1/4 h ou de 1 h avec une autonomie de 40 j ou de 160 j.
  Par une liaison RS 232, on sort tout de suite un graphe de l'évolution tel que représenté à la figure ci-après.

0000	10H	
0009	09H	
0003	08H	
0000	07H	
0000	06H	
0000	05H	
0000	04H	
0000	03H	
0000	02H	
0000	01H	
0000	00H	
0000	23H	21/07/84
0000	22H	
0000	21H	
0000	20H	
0000	19H	
0000	18H	
0000	17H	
0000	16H	
0000	15H	
0000	14H	
0000	13H	
0000	12H	
0000	11H	
0000	10H	
0000	09H	
0000	08H	
0000	07H	
0000	06H	
0000	05H	
0000	04H	
0000	03H	
0000	02H	
0000	01H	
0000	00H	
0000	23H	20/07/84
0000	22H	
0000	21H	
0000	20H	
0000	19H	
0000	18H	
0000	17H	
0000	16H	
0000	15H	
0000	14H	
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0000	07H	
0000	06H	
0000	05H	
0000	04H	
0000	03H	
0000	02H	
0000	01H	
0000	00H	
0000	23H	19/07/84
0000	22H	
0000	21H	
0000	20H	
0000	19H	
0000	18H	
0000	17H	
0000	16H	
0000	15H	
0000	14H	
0001	13H	
0000	12H	
0000	11H	
0000	10H	
0000	09H	
0000	08H	
0000	07H	
0000	06H	
0000	05H	
0002	04H	
0000	03H	
0002	02H	
0000	01H	
0000	00H	
0000	23H	18/07/84
0000	22H	
0000	21H	
0000	20H	
0022	19H	
0208	18H	
0000	17H	

La consommation actuellement est de 2 mAh. Une nouvelle version en cours de développement aura des performances largement accrue avec une capacité de mémoire quadruplée et une consommation de 50 μAh.

- Stations météorologiques :

  Le même développement permet de recevoir 8 capteurs analogiques et 8 entrées digitales. Ce qui conduit à un enregistrement de l'ensemble des paramètres nécessaires au calcul de l'évapotranspiration avec établissement de moyennes, de maximum et minimum, etc....

CONCLUSION

Les développements actuels de la micro-informatique tant hardware que software ouvrent aux hydrologues de nouvelles perspectives tant dans le domaine de la saisie de l'information (résolution, affinement des pas de temps, synchronisation des mesures) que du transfert de l'information et d'une aide à la prise de décision dans le domaine de l'étude des ressources en eau en général et de l'hydrologie en temps réel en particulier.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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THE ROLE OF GROUNDWATER IN RURAL DEVELOPMENT IN EGYPT

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ABSTRACT

The overall strategy for groundwater development is based on the government policy for the movement of population from the narrow strip bordering the Nile and improve the living condition in the rural communities. Groundwater is a major component of the environment and determine whether or not life is possible under arid climatic conditions in the Egyptian deserts.

The expected increase for both irrigation, domestic and industrial waters, call for more attention to the exploitation, management, proper use and control of groundwater. The planning for groundwater include also its conjunctive use with the available surface water since it will lead to:

- Improve the drainage efficiency.
- Reduce the groundwater levels.
- Increase the land fertility & in turn increase the national production.
- Finally it will lead to the rural society development as long as the groundwater is used in an effective economic way in different aspects of life as; municipal and drinking water, irrigation, industry and other uses according to the priorities layed out by the planners in these societies.

This paper discusses generally the overall strategies for groundwater development in both the Nile Valley, the deserts and its role for conservation of the environment.

INTRODUCTION

The groundwater represents the keystone for any development activities in the arid areas, specially in deserts where it is the lonely water source in these areas. Therefore the different activities related to groundwater investigation, exploitation, conservation and management is the main issue to prove the development goals and to develop and extend the rural areas.

Since the dawn of history it was well understood that a permanent water resource is essential for fulfilling the society requirements in all fields of life. Thus they started to investigate and develop the groundwater where the rain is scarce and no rivers or streams are existing, then they started to exploit and manage the groundwater untill we reached the sophisticated methods of today in these fields.

In Egypt the overall strategy for groundwater development is based on the general policy of moving population away from the narrow strip bordering the Nile to improve overall living conditions specially in the desertic areas.

At present about 20 million citizens use groundwater for drinking purposes in the Nile valley and its Delta. Also about 125 thousand acres are irrigated by groundwater. In 90% of the Egyptian desert, where new cities have been established the only source of fresh water is the groundwater reservoir. The different studies in the field of groundwater have been conducted in three main lines:

1. The Nile valley and its Delta, where the studies have been completed and the results have shown that 5×10^9 cubic meters can be used safely either directly or by conjunctive use to save surface water to establish new rural societies specially in desertic areas.
2. Studies in the western desert where the Nubian sandstone aquifer is the potential for the development process and to prevent desertification effect in the areas bordering the Nile valley and its Delta.
3. Groundwater studies in Sainai peninsula, which is the main foundation for the overall development process.

Planning for groundwater development and use in Egypt is given a great priority in order to prove the Egyptian goals in developing their societies, specially when considering the limited amount of surface water and the urgent need for new water resources to balance the rapid increase in Egyptian population.

GROUNDWATER DEVELOPMENT IN THE RURAL AREAS IN EGYPT

The groundwater reservoir in the Nile valley and Delta underlies over 25000 square kilometers of fertile Egyptian soils. The valley in Upper Egypt is deeply incised into rocks ranging from cretaceous to Eocene age and is filled to varying depths with Quaternary alluvium with thickness ranging between 50-250 m. while the Nile valley begins to open out into a triangular alluvial Delta at a distance of 20 kms.

North-West of Cairo. The bulk of the aquifer under the Delta consists of deltaic deposits of 200-700 meters thick. This alluvial formation is in general along the Nile Valley and Delta, is composed of a lower portion of unconsolidated gravel, sand silt and overlying layer of less permeable clay and silt with some sand, which is an aquitard (clay cap). The aquitard is also important as it retains the moisture for plant growth and permits the drainage of surplus water to the overlying alluvial aquifer which can be considered as partial confined aquifer but with water table conditions at the fringes of the valley and Delta.

Groundwater is extensively used as auxiliary source for irrigation water supply and as city water supply in towns and villages in the Nile Valley and Delta. Tables (1) and (2) show the quantity of groundwater used for irrigation and drinking purposes in each governorate of Upper Egypt and Delta respectively, refering to these tables the total quantity extracted from groundwater reservoir underlying the Nile Valley is 1.4 milliard cubic meters per year, of which 7% is used for drinking purposes. In Delta, the total quantity reaches 1.6 milliard cubic meters of groundwater per year, of which 30% is used as drinking water. Recent studies carried by the Research Institute for Groundwater has indicated that additional amount of 1.5 milliard C.M/year is planned to be withdrawn from Upper Egypt plus an additional 0.5 milliard C.M/year from the Nile Delta during the coming ten years to fulfill the future water demands. This dictates to the establishment of a groundwater management plan.

The achievement of conjunctive use of the surface and groundwater reservoirs of the Nile Valley must be the long-term strategic objectives of water management. It is considered that such conjunctive use would be progressively developed. The following objectives are thought to be attainable:

Table No. (1)

Governorate	Annual Extraction MCM (U.EGYPT)		
	Irrigation	Domestic Water	Total
Beni Suef.. ..	7.00	21.31	28.31
Minya	247.00	22.72	269.72
Assiut	147.00	10.83	157.83
Sohag	584.00	26.62	610.62
Qena	342.00	10.00	352.00
Aswan	-	8.85	8.85
Total	1327.00	100.33	1427.33

Table No. (2)

Governorate	Annual Extraction MCM (DELTA)		
	Irrigation	Domestic	Total
Cairo	149.50	89.00	238.50
Giza	19.00	44.25	63.25
Qalyubia	53.02	58.12	111.14
Sharkia	114.17	45.67	159.84
Dakahlia	15.50	26.05	41.55
Gharbia	9.00	128.49	137.49
Minufia	40.00	69.67	109.67
Tahrir	597.00	16.00	613.00
Behera	4.00	10.26	14.26
Total	1001.19	487.51	1488.7

a. Increased reuse of irrigation water by pumping from groundwater.
b. Making water available for new lands at the desert fringes of the valley.
c. Control and prevention of water logging and salination through vertical drainage aspects of the approach.
d. Reduction of water losses to the sea.
e. Operation of High Dam and Aswan Dam freed from hydroagricultural constrains so as to maximize power generation.

An early stage in equipping the groundwater reservoir would be to supply irrigation for reclaimed lands bordering the presently irrigated areas. Such water would be drawn from under old irrigated lands, a further advantage would be the resulting enhanced drainage of old lands. The drilling of wells to pump the groundwater, the necessary changes in irrigation techniques and water management and the organizational changes will all require considerable time. Two pilot areas each of 5000 feddans have been chosen one at the centre of Delta and the other at Middle of the Valley in Upper Egypt. These two pilot areas will be equiped with 70 tubes wells at each area and electric deep well pumps be installed. The evaluation of the practices at these areas will give guide line for planning and design of ground and surface water conjunctive use projects at the Nile Valley and Delta.

Groundwater plays an important role for conservation of the environment, healthy living conditions depend in great measure on the control of the water component and availability of good quality water which is a major factor in improving and maintaining high health standards. Attention at strategic level must be in cooperation with public health authorities. It is desirable to concentrate on continuing subjection eradication of major water borne diseases; bilharzia and malaria. Groundwater provides many amenities of life, particularly in such a hot dry climate. Encouragement should be given to use of groundwater in development of such features as shade trees, parks, gardens and pools.

Planning stages must include measures to remove the danger of rising groundwater levels which have been noticed in many villages in the last few years and which may endanger many structures ranging from monuments to present dwelling places. Also the groundwater reservoirs should not become the public sewers of villages.

GROUNDWATER DEVELOPMENT IN EGYPTIAN DESERTS COMMUNITIES

The strategy for indigenous waters of the Egyptian deserts can be considered with respect to water resources, to water uses and to the schedule under which resources and uses are brought together by development and management. The basic strategic consideration for indigenous water must be reiterated. As yet, the indigenous water resources are uncertain as to their quantity, their quality, their reliability and their replenishment. These must be ascertained by surveys and investigations before they can be allocated to different uses, and the short fall between supply and demand filled from other sources. To East Desert water strategy for indigenous waters should be based on the idea that natural renewable water resources are not immense and will not serve for very large scale development. Mining of groundwater will provide additional waters. In some areas, as along the Red Sea coast, heavy extractions may mean exhaustion of the resource in say 20 years. In other areas, such as from Nubian sandstone aquifer for irrigation, it may be hundreds of years before the economic limit to extraction is reached. Groundwater from Nubian sandstone is expected to be of a quality suitable for irrigation.

A groundwater from Nubian aquifer can be extracted to supply the development at the western desert and oasis. Experimental work on the uses of non-conventional energy sources should continue or be initiated, solar power for pumping groundwater or for small-scale demineralization is of interest. Wind power may be used for lifting stock water. Gravity may bring groundwater to surface through infiltration galleries. Other sources of power are being investigated throughout the world to reduce dependance on non renewable fossil.

Table No. (3)

Groundwater basin	Annual extractions in million Cu.M Year 1980	Annual Potentials in million Cu.M. Year 2000
1. West Desert		
Kharga........	90.0	109.4
Zayat	-	14.2
Abu-Tartur....	-	21.8
Dakhlia.......	198.0	416.9
West-Mawhoob..	37.4	37.4
Abu-Muncar....	15.0	47.0
Farafra.......	1.3	81.3
Karawein......	-	(192.0)*
Bahariya......	34	122.4
Siwa..........	43	( )*
2. East Desert		
Wadi Qena.....	-	(50)*
Wadi Lqeita...	0.6	(25)*
Wadi Abbadi...	-	(25)*
Wadi Nattash..	-	(50)*
Coastal plains	1.0	( )*
3. Sinai		

* Further studies and investigations are needed.

CONCLUSIONS

1. Water resources determination and development must be directed towards improving the standards of living of the population, and to facilitate movement of people from the overcrowded strip bordering the Nile river to new lands offering potential for expansion, wider use of natural, and on improved quality of life for the resettled communities.
2. Increased and more use of water in consumptive and non-consumptive supply for the production of food, power, services and amenities will call for increased skills and integration in water management. New tools for water management will be required and some operations will be computerised.
3. Water is a major component of the environment and determines whether or not life is possible under arid climatic conditions. Water can contribute much to the environment, in irrigated agriculture and in urban and rural amenities. Conversely improper use and control of water can lead to deterioration of the environment with danger of increasing incidence of such diseases, the possibility of discharges of untreated sewage and industrial waters, the prospect of water logging and increased salinity of soils rising groundwater levels can endanger ancient buildings of major historical importance. Water weeds can spread uncontrolled within man-made lakes and waterways.
4. Precedence must be given to management, with the objective of optimising their value through increased reuse, and the

conjunctive use of the surface and underground reservoirs.

5. Water resource surveys for indigenous water must be carried out and pilot and full development of groundwater for urban and industrial uses has priority in time and should follow or overlaps the survey of the groundwater potential.

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OPTIMIZATION MODELS FOR REGIONAL WATER PLANNING IN NEW RECLAIMED LANDS IN EGYPT

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ABSTRACT

Two mathematical models were developed by means of which the coordination of regional water resources supply and planning demands for the new reclaimed lands in Egypt is reached. For the old lands, the model is a water supply which allocates water at various subareas to satisfy the demands at a minimum cost. For new lands, the model is a water demand one that allocates the cropped area at a maximum net benefit. The coordination between the two models gives the optimization of the net benefit for the development of the old and new land as one unit. The models were applied to El-Sharkia governorate in Egypt.

1. Introduction

Egypt is now facing a major challenge as how to increase the the growth of the agricultural production to meet the future food requirements and to reduce expenditures for imported food. Despite advances in industrialization and increased urbanization, the agricultural sector still accounts for 50 percent of the total population, 47 percent of the total employment, about 30 percent of the Gross National Product, and 80 percent of export earnings.

With these stark realities as a background, this study focuses on the regional agricultural planning sector, examing the basic constraints of land, water (quantity and quality), and the limited alternative cropping patterns. The reclamation and development of new lands have been a major aspect of agricultural policy in Egypt. The increased quantity of irrigation water available from the High Dam, mounting population pressures in old land areas, and a substantial number of land less farmers, all encouraged expanding the agreculturaI land base. Frequently these areas were selected without sufficient analysis and evaluation of their ecomic feasibility as far the overall allocation of the investiment between the old and new lands. Thus the purpose of this study is to introduce an integrated model by means of which the optimum ultilization of all the available water resources (surface water, ground water and drainage water) to meet spatial and temporal demands at the least cost for the old land is reached. The optimization problem arises due to the difference between the actual water distribution and the suggested water distribution using all the available resources at a minimum cost. The fixed areas and cropping patterns associated with the old land suggest that optimal allocation is attained through minimizing the overall cost, thus maximizing the benifits. For the new lands, some possible crop patterns can be selected and the coresponding areas can be reached through the optimization of the net benefit. One of the main objectives of this endeavour is to determine the quantities and qualities of water which can be saved from the existing old land to serve as an imput to the new reclaimed lands. Another important goal of this study is to investigate the effect of the changes in water quality resulting from the use of drainage water on the yields gained from the old and new lands if considered as one integrated unit in the planning process.

Two mathematical models are incorporated for this purpose. The first model is applied to the old lands and its objectiv is to minimize the cost of water allocation satisfying the demands and saving water for the new lands. This model is a water supply model. For the new land planning, the second model is developed which is a water demand model that allocates the cropped areas at a maximum net benefit. The agricultural revenue is the only benefit considered in this case.

The derived models were applied to El-Sharkia Governorate east of the Nile Delta, Egypt. Suggestions for implementation of new planning scheme are offered resulting from the application of the models.

2. Mathematical models formation

2.1. Concept of the models

In these models it is assumed that the existing areas to be irrigated and its crop pattern are previously decided and remain unchanged during the planning period. This assumption will be relaxed, however, for the new lands.

The model decision variables are the seasonal discharges and capacities of mixing stations for the existing lands and the area location and its cropping pattern for the new lands. For the purpose of analysis, the region is divided into subareas each contains all the element pertaining to the water supply and water demand. The supply sources are:(i) canals, (ii) drains and (iii) groundwater. The demand elements are:

(i) agriculture and (ii) municipal and industrial uses. Two seasons for water requirement of main crops are considered; the winter season (September thru February) and the Summer season (March thru August).

For new lands area, the supply source is the water saved from the old lands. The agricultural revenue is the only benefit considered in this case. The same two seasons are selected for the new land analysis.

2.2. Basic assumption

The foll wing assumptions are made for model formulation:

(i) The horizon time given as T years.
(ii) The cost function remains stationary over time T years.
(iii) The capital cost of irrigation works, new mixing stations and land developments are to be borrowed for a period t_1 with an interest rate e.
(iv) The discount rate r remains constant during the time T.
(v) For the new lands, the cultivation will commence after time t_1, and all the new lands are reclaimed in the same time.

2.3. Old agricultural land's model

For the old lands, the objective function is to minimize the total costs, thus maximizing the net benefit. The function may be expressed as:

$$\text{Min } Z_1 = COMS + CMC + CMD + CFL + CPW + (OMR + E)$$

where

COMS = The capital cost of new mixing stations.
CMC = The maintenance and operation cost of canals,
CMD = The maintenance and operation costs of drains,
CFL = The cost on fram level,
CPW = The cost of pumping wells for groundwater,
OMR+E = Operation, maintenance, Replacement and Energy Cost of mixing station (existing and new)

2.4. New reclaimed land's model

For the new lands, the objective of the model is to maximize the net benefit of the agriculture production. Mathematically, this is given by

$$\text{Max. } Z_2 = APR - CELSC - CMC - CESD - CMD - CFL - CLD$$

where

APR = Agriculture production revenue

CELSC = Cost of construction, lining and irrigation structure of canals.

CMC = The maintenance and operation cost of canals,

CESD = Cost of construction of drain,

CMD = The maintenance cost of drains,

CFL = The cost on farm level, and

CLD = Land development costs.

2.5. Constraints

The constraints are the conservation of mass for surface and ground water, the upper limit for pumping of ground water, the water quality (salt concentration), the maximum allowable withdrawal from drains. For new lands other constraint are to be considered as the area budget, crop yield and the limitation on crop water requirements.

3. Planning for El-Sharkia governorate water resources and the proposed land reclamation

The models formulated are applied to El-Sharkia Governorate and to the new proposed land in the north eastern side of the Governorate. The benefit-cost ratio for the combined existing lands and the new developed area are taken as the criterion for obtaining the optimum allocation of water resources in the old lands as well as the crop pattern and its corresponding cultivated areas for the new lands.

El-Sharkia Governorate is located east of the Nile Delta. The total command area (the present cultivated land) of the Governorate is 547373 feddans divided into 10 irrigation sectors. The proposed reclaimed areas are located to the east and north of El-Sharkia Governorate. Its total area is about 282,000 feddans divided into 5 subareas.

The main sources of Irrigation water are the Ismailia Canal where El-Saidiya Canal and El-Wadi El-Sharki Canal bifurcate and El-Riyah El Tawfiki flows with tributaries bahr Muweis. Drainage water is used after mixing with fresh irrigation water at two sites. The first is El-Wadi irrigation pumping station at El-Zagazig with annual discharge of 450 million m^3 and the second is the Hanout irrigation pumping station at Kafr Sakr with annual discharge of 274 million m^3.

The use of groundwater for irrigation and municipal supplies constitutes an additional source of water. The annulal amount used of this water is 110 million m^3 in the year 1978.

For the new reclaimed areas the main source of irrigation water is the water to be saved from the existing irrigation system.

The major water demand in El-Sharkia Governorate is for agriculture uses. The municipal and industrial water demands are relatively small which represents about 3% of the agriculture uses.

4. The outcome of the models.

The model was applied at first to the existing cultivated areas to determine the water allocation for different sectors and to determine the quantity and quality of water which can be withdrawn from the region. The saved water from the existing lands is considered as the inflow to the new land. The model was then applied to the new land to get the optimum cropping pattern together with the area allowed to each crop. The computation was conducted using the 1900 I.C.L. series linear programming system known by Mark 3. This program is available in the Scientific computer Research Center of Cairo University. This package is designed to suit medium and large linear programming problems. The package implements the revised simplex algorithm for solution, and can perform sensitivity analysis and parameteric programing.The results showed that the maximum available amount of water to be withdrawn from the region is 1190 x 10^6 m^3/year with salinity concentration of 1000 p.p.m. For each run, the marginal cost of the water withdrawn was calculated. It was defined as the difference between the cost of saved water and regional cost with no output divided by the quantity of water saved. The marginal cost versus quantity of water withdrawn is represented in seven different steps as represented in Fig(2). The diagram determines the price of the water to be withdrawn for the development of the new land. At outflow 916 m x 10^6 m^3/year and up to the maximum saved water, five sectors used drainage water instead of the three sectors reported for the present situation. Water for municipal and industrial requirements is withdrawn from groundwater basin in most of the subareas. For the new land development seven computer runs were carried out with different input flows to the land taken as the quantity of water saved from the existing land. Inflow values incorporated both the quantities and qualities of water. The outcome of the runs revealed for (Case I) that the model selects the crops which have the highest revenue. For the winter season (September to February) the long Season Clover was chosen while vegetable are selected for summer season (March to August). The Ministry of Land Reclaimation identified certain crops and the percentage of area to be occupied by each crop considering socio-economic factors (Case II). These crops are short season Clover, Long Season Clover, Bearly, Vegetables and Bean in winter and in summer Cotton, Rice, Maize and Vegetables. Under this second case, the model was applied to get the benefit-cost ratio. Table 1 shows B/C for both new and old lands for case I and Table 2 shows B/C for the combined new and old lands for case II.

Table 1: Cost, benefit and B/C ratio for the combined existing and new land without constraint on crop pattern for the new lands (Case I).

Amount of inflow in m.m^3/year	Cost x 10^8 L.E. C	Benefit x 10^9 L.E. B	B/C
Zero	8.37831	1.95416	2.33
145	8.57298	2.04945	2.39
460	9.00186	2.25975	2.51
916	9.63284	2.56773	2.66
1042	9.81406	2.66522	2.72
1070	9.85389	2.69016	2.73
1126	9.93568	2.70267	2.70
1190	10.03860	2.68030	2.67

Table 2: Cost, benefit and B/C ratio for the combined existing and new lands with constraint to the crop pattern of the new lands (Case II).

Amount of inflow in m.m.^{3}/year	Cost x 10^8 L.E. C	Benefit x 10^9 L.E. B	B/C
Zero	8.37831	1.95416	2.33
145	8.57291	2.04945	2.39
460	8.94075	2.06323	2.31
916	9.28749	2.07218	2.23
1042	9.37290	2.07183	2.21
1070	9.39184	2.00151	2.20
1126	9.43297	2.06946	2.19
1190	9.48796	2.06727	2.18

5. Conclusions

The application of the proposed models showed that three new-mixing stations are to be put into operation for the old and new land developments. The change in water salinity after mixing fresh and drainage water is within the tolerence of the crops chosen in new lands. About 227000 feddans of new area cultivated by long season clover during winter and vegatable during summer gives the maximum net benefit.

6. References

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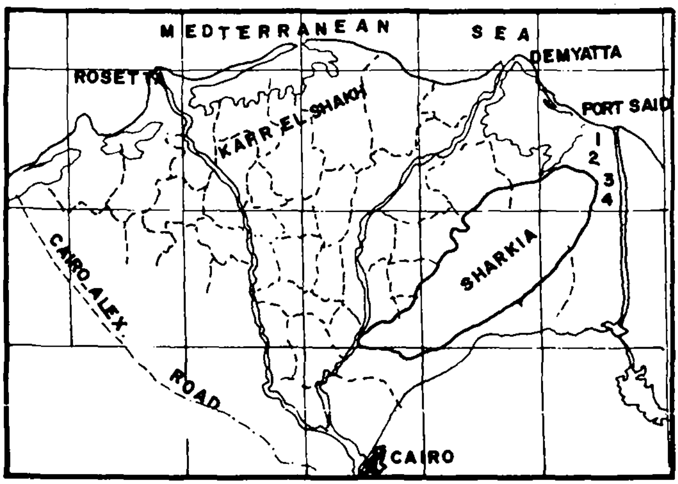

Fig.(1) EL_ SHARKIA GOVERNORATE AND THE PROPOSED NEW LANDS

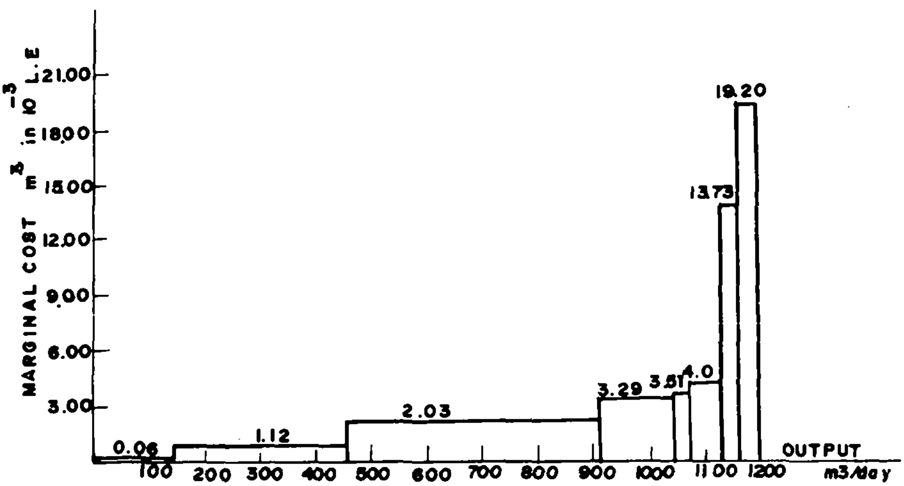

Fig. (2) THE MARGINAL COST VERSUS WATER OUTPUT.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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EGYPT WATER MASTER PLAN
INSTITUTIONAL BUILDING FOR DECISION MAKING

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ABSTRACT

The Ministry of Irrigation together with the World Bank under the United Nations Development Program began a Master Plan for Water Resources Development and Use in October 1977.

Phase I of the project terminated in March 1981 and the results were recorded in a main report and twenty appendices.

Water Planning is a continuous process. Changes in demands, supply, technology, costs, prices, national goals take place. Alternations in short and long range water plans must be made to accomodate these changes.

Therefore, a second phase of the project started in January 1982 scheduled for three years. While Phase I primary emphasis was on the development of planning tools and resources, skills and capabilities of project staff, the primary emphasis in Phase II is to develop the Water Planning Group (WPG) as nucleus for a water planning and coordination organization. WPG is to act as the technical secretariat of an Interministerial Water Planning Committee (IWPC) under the chairmanship of H.E. the Minister of Irrigation, the custodian and manager of the country's water resources, and the membership of representatives from all ministries and authorities using the water resources. The primary function is to provide a broad, macro-economic prospective of the planning and programming of projects envisioned by each ministry concerned with water uses, in a cross-sectoral and multidisciplinary approach.

This paper discusses the role of water planning in Egypt and concentrates on the objective of institution building for decision making.

1. INTRODUCTION

Egypt has a rapidly growing population, a growing industrial complex and a policy to try to acheive food self sufficiency.

Consequently its water requirements are growing. Water is required for expanding industry, growing urban centers and towns, and expanding irrigated agriculture. Several government agencies have functions and programs and develop projects which impact on Egypt's water supply. Egypt's water supply, although sufficient now, will be exhausted in the near future at the presently planned rate of growth for industry, agriculture and urban consumption. Funds for implementing projects to utilize existing supplies and projects for obtaining new supplies are not unlimited, and much of the funds for water development are borrowed or granted from sources outside of Egypt, such as international banks and foreign governments. These fund providers want some assurance that water projects are planned, investigated and coordinated properly before they loan or grant funds.

2. INITIATION OF WATER MASTER PLANNING IN EGYPT

The Ministry of Irrigation together with the World Bank under the United Nations Development Program began a Master Plan project in October, 1977. This project was jointly financed by the Ministry of Irrigation, the United Nations Development Program, and the World Bank. Three expatriate water resources specialists and a few engineering consultant firms worked with a small national planning staff during Phase I.

2.1 Objective

The objectives stated at the beginning of this planning effort were :

1. Conduct a comprehensive study of the present and future water requirements, and sources of supply. Conduct a scientific and economic analysis for the different alternatives for developing water resources.
2. Develop resources policies for agricultural development and food security, industrial growth, municipal requirements, electric energy generation, and navigation.
3. Study the impact on water quality from waste discharges from agriculture, municipalities and factories to determine appropriate treatment and management programs as well as the potential for reuse.
4. Determine the size of investment needed overtime to develop and use water resources.
5. Develop an integrated operation system for coordinating hydro-electric energy generation with other uses on the Nile system. Operating criteria for the Aswan High Dam is very significant in this system.
6. Develop mathematical models and other planning tools for continuous water resources planning. These planning tools will greatly assist in developing and implementing national water resources policies.
7. Train national staff members in planning procedures to enable them to continuously conduct and update national water planning.

2.2 Phase I Water Master Plan Project

The first phase of this United Nations Development Program Project lasted 42 months. The results were recorded in a main report, an executive summary and twenty technical appendices. Accomplishments of Phase I are as follows :

2.2.1 Project Information System

A computerized data bank was established to store and process data. It is composed of three sections, storing agro-economic data, irrigation network data, and drainage system data.

2.2.2 Potential water supplies

A list of potential future water supplies for Egypt were summarized. Studies were conducted to estimate the quantities of water available from these sources and their costs. The potential sources included Upper Nile marsh channelization projects, (Jonglei I, Jonglei 2, Machar Marshes, Bahr El Ghazal Swamps), drainage water, municipal and industrial wastewater, ground-water, salt water conversion and water saved through better water system operation and maintenance procedures.

2.2.3 Water requirements and policies

Future water requirements were estimated for municipal, industrial, navigation and agricultural uses. Estimates were made to the year 2000. Three levels of development of irrigated new lands were estimated. Costs and water quantities were estimated. Policies concerning priorities of water use were formulated.

2.2.4 Water quality

A study of water quality was conducted. Its results describe present water quality in Egypt and considers future problems and requirements related to water quality for public health, ecology and irrigation.

2.2.5 Mathematical models and programs

The project developed mathematical models and computer programs as planning tools for studying water resources in Egypt. Models and programs developed include an Agro-Economic Model, Upper Nile Water Routing Model, High Aswan Dam Simulation Model, Dynamic Programming Model for Lake Nasser, Equatorial Lakes Model and a Distribution Model for the Lower Nile system.

2.2.6 Future water scenarios

Three scenarios for water resources development defining water requirements and supplies over time were developed. Estimates of costs were also made. The scenarios assumed municipal and industrial requirements the same for each scenario. Irrigation use was estimated at differing levels from 50,000 feddans of new land development per year to 180,000 feddans of new land development per year.

3. WATER MASTER PLANNING - A CONTINUOUS PROCESS

Water planning is a continuous process. Changes in demands, supply, technology costs and prices, national goals and national security occur. Alterations in short and long range water plans must be made to accomodate these changes.

3.1 Present National Situation

The population of the country is growing at a very high rate of 2.8% per year. The country has about 5.8 million feddans of irrigated land and a potential of another 2.8 million feddans of desert land suitable for irrigation. The present 5 year plan (1982/83 - 1986/87) aims at an overall growth rate of not less than 7.5% annually.

Commodity production is to be the basis of development. The development goals by sector are :

Industry	9.0%
Agriculture	2.8%
Productive Service	6.1%
Housing	12.5%
Social Services	8.3%

The plan aims at decreasing imports, increasing exports and lessening the balance of payments deficit and need for borrowing from abroad. The specific goals of the present 5 year plan in this respect are :

a. Decrease the deficit in Balance of Payments from the present L.E. 2 billion to L.E. 1.6 billion.

b. Increase the ratio of exports to Gross Domestic Product from 26.1% to 27.1%.

c. Decrease the ratio of imports to Gross Domestic Product from 38.7% to 36.6%.

d. Provide 366,000 new employment opportunities.

e. Raising the standard of living-collective consumption which represents the free services rendered by the state to the citizens will increase by 8.3% throughout the 5 year plan.

These goals will have an impact on water requirements and water management and decision making.

3.1.1 Cost of development

The present five year plan estimates an investment of L.E. 25.8 billion for public sector projects and about L.E. 8 billion for private sector projects. Of this total cost funds for the construction, operation and maintenance and administration of water related programs for agriculture, navigation and irrigation alone amount to L.E. 2.5 billion. Municipal and industrial water costs are not included in the L.E. 2.5 billion.

Water related proposed projects and operation and maintenance of various water systems or functions are managed and controlled by several federal and local government agencies.

3.1.2 Government organization

Key federal agencies which manage and carry out functions concerning water are listed below. These listed ministries have many organizations within them which manage more specialized functions.

Ministry of Irrigation

This agency controls the Major Nile system and its irrigation and drainage structure. It schedules water releases from Aswan, approves diversions from the system and recently was given authority to implement the new national water quality legislation. It is also in charge of implementing a large program for installing underground drainage systems in a large proportion of Egypt's irrigated lands.

Ministry of Housing and Land Reclamation

This agency is primarily responsible for developing new land irrigation projects and the infrastructure accompanying these developments. It is also responsible for new Potable Water and Sanitary Sewerage projects.

Ministry of Electricity and Energy

This agency is responsible for electric projects including fossil fuel plants which require cooling water. This agency is involved in Hydro-Electric facilities and operates the Hydro electric plant at Aswan Dam.

Ministry of Industry

This agency is responsible for industrial projects which use water and discharge effluent into water channels.

Ministry of Transportation

This ministry is responsible for navigation and is therefore concerned about locks, dredging, proper navigation flows and navigational effects on river and canal banks.

Ministry of Health and Environment

This agency is concerned with environmental effects of water projects, water quality criteria and research in environmental issues.

Ministry of Agriculture

This ministry is concerned with all the various aspects of agriculture. Since virtually all agriculture in Egypt is irrigated agriculture, this ministry is involved in water issues such as crop consumptive use, drainage requirements, and proper irrigation schedules.

Ministry of Planning

This agency is the nation's economic planning entity and has much authority in fund allocation, project selection and timing and the setting of national economic goals.

These agencies affect water requirements and supplies to various degrees and in various ways.

3.1.3 Water use and supply

Existing water supplies presently developed for use in Egypt are :

Source	Billion m^3 per year
1. Nile River allocation available at Aswan Dam.	55.5
2. Drainage Re-Use	2.3
3. Ground-Water	2.9

Potential new water supplies can be obtained from further drain water reuse, ground water development and Upper Nile projects. The Jonglei I project in the Upper Nile will provide another 2.4 billion cubic meters per year of water around 1985. Potentially developable additional drainage water re-use and groundwater use are 6 to 7 billion cubic meters annually and 2 billion cubic meters per year respectively.

3.2 Phase II Water Master Planning

Because water planning is a continuous process and given the present national situation, the Ministry of Irrigation and the World Bank agreed to initiate Phase II of Water Master Planning in Egypt under the United

Nations Development Program.

Phase II began in January 1982 and is scheduled for three years. Two World Bank expatriate employees and two USAID expatriate economists are advisors for plan development and training.

3.2.1 Objectives

The primary emphasis of Phase I was on the development of planning tools, and resources and skills and capabilities of the project staff. The main output of the project, namely the Final Report, concentrated on the long range 20-year planning prospective. In Phase II while still continuing to improve the planning tools developed in Phase I, and updating the data base, the primary objective is to give emphasis to tangible and utilitarian outputs principally related to the short term planning horizon. Phase II outputs are to be designed to be immediately useful to high level policy makers and planners, to project planners and to those responsible for approving and adopting project and program investments.

Phase II is to develop the Water Planning Group (WPG) as a nucleus for a water planning and coordinating organization. WPG is to act as the technical secretariat of an Inter-Ministerial Water Planning Committee (IWPC) under the chairmanship of H.E. the Minister of Irrigation and custodian and manager of the country's water resources. The primary function is to provide a broad, macro-economic perspective of the planning and programming of projects, envisioned by each Ministry concerned with water use, in a cross-sectoral and multidisciplinary approach.

The responsibilities of the WPG include :

1. Maintaining an up-to-date inventory on all water and agro-economic data in Egypt.
2. Developing, maintaining and updating, as necessary, a project portfolio on every water related project.
3. Providing a planning framework for decisions on policies and project priorities.
4. Reviewing of all plans and projects related to water within the framework of a dynamic water plan.
5. Assisting in the preparation of continually updated investment package of viable water related projects for financing and implementation within the framework and time schedules of an overall dynamic plan.
6. Monitoring and evaluating implemented projects, when projects' data became available, and capability built up.

3.2.2 Framework Plan and Planning Tools

Phase II of the Water Master Plan Project differs from Phase I in that it emphasizes decision making processes, institution building and immediately useful evaluations of proposed water projects in the five year plan.

In order to set up a process for coordinated decision making concerning the national water situation, H.E. the Minister of Irrigation issued Decree No. 322/1981. The purpose of this decree was to establish an

Interministerial Water Planning Committee (IWPC) representing all the national agencies involved with water. This group is supposed to :

1. Propose and approve strategies adopted by the WPG for achieving a comprehensive plan of water resources development.
2. Supervise and direct the performance of the WPG.
3. Study water related policies, plans, programs and projects from the various member agencies and recommend approval or modification within the overall national framework plan.
4. Monitor implemented and on-going water related plans projects.
5. Member agencies are to submit data, water related plans and projects studies to the WPG for evaluation so that the IWPC can make decisions concerning these projects, and plans within the overall framework water plan.

Later he updated this decree with Decree No. 142/1982 which is the same as the first decree except for the addition of two more member agencies.

The IWPC is supposed to meet monthly and the decision making process envisioned is as follows ; (This process is also shown on Chart attached).

A national long range framework water plan and specific water policies are to be developed by the WPG under the broad guidance of the IWPC. The coordination of all water related projects being proposed by all the federal agencies is to take place through a process of agency submittal of project studies to the WPG for review and evaluation. One of the WPG's primary tasks is to measure the affect of these proposed projects on the total water supply and continuously update the national water balance supply and demand. Other tasks relate to the economic feasibility of the projects and the total costs. The WPG is to review the economic study made for each project by the submitting agency. The WPG is to review the methods used and reasonableness of the data used. The Water Planning Group would then rank the projects in the order of their economic feasibility and also show the total costs of all projects as they are scheduled over time. The IWPC can then use the results of WPG reviews to make decisions on :

1. Water supply - when planning for new supplies should begin and which new supply source should be developed first.
2. Approval of projects - which projects meet proper planning methods and fall within national policies and guidelines, which projects are most economically feasible.
3. Allocation of funds - when funds do not cover all proposed projects, select the most needed and most economic projects.

The IWPC can then develop a coordinated annual or five year national water plan to be submitted to the Ministry of Planning for inclusion in the overall 5 year national plan. Once the Ministry of Planning approves this plan and its projects, it can be funded by the Government and subsequently implemented by the sponsoring agencies (IWPC members).

4. CONCLUSIONS

4.1 National Macro Water Planning Beneficial

National macro water planning is a beneficial and continuing process.

Benefits include providing coordination of activities involving water; establishing water policies for guiding water use and development; providing early warning concerning the depletion of existing supplies and the need to search for and develop new supplies; introducing modern technology for the planning, construction and operation of water projects; obtaining the most beneficial water program within the financial budget allocated to the national water program, and establishing uniform and adequate project planning criteria which enhances the projects chances for funding by the national government, international banks and foreign countries.

4.2 Phase I of Egypt's Water Master Planning - A Good Start

Phase I of the nation's Water Master Planning effort is a good start. It established some criteria, developed a data base, determined inadequacies in certain data and data gathering processes, identified specific water activities that should be improved or undertaken for more efficient water management, provided a preliminary estimate of the exhaustion of existing water supplies under various levels of development and developed some modern technological tools for water planning.

4.3 Egypt's Growth and National Goals Require Careful Husbanding of its Water

The country's rapidly growing population, five year plan development goals and its stated policy to become food self sufficient require careful management of its water resources. Meeting these national goals will require more water of appropriate quality. Careful planning, improved operation and maintenance of water systems, conservation programs and careful investment will be required to meet future projected water requirements.

4.4 Phase II of Egypt's Water Master Planning is a Necessary Continuation of National Water Planning

National Water Planning is a continuous process and Phase II is a proper step to plan for the development of the nation's waters. Phase II is updating a long range framework plan developed in Phase I, improving and expanding planning tools developed in Phase I and attempting to develop a Master Water planning institution and decision making process which will provide decision makers with the information, analysis and studies required to make sound water development and investment decisions.

4.5 Consideration of Alternative Planning Institutions and Decision Making Processes May Need to be Examined in Light of Phase II Experience

As described, the institution and decision making process envisioned for Phase II has not really been evaluated. However, it may prove to be a difficult process. It is especially difficult when group members each have different water related programs to implement and all compete for limited funds. It requires a lot of good will from all members to provide all their data and project studies for review by a Water Planning Group and to agree to priorities established by the review and analyses which occur in the group. Coordination of water activities is extremely beneficial especially where many agencies have responsibilities affecting the nation's water resources. A national water policy framework and plan are highly desirable as a guideline to all agencies as they carry out their responsibilities.

A visible and authoritative institution may be required to realize coordinated water related activities, adherence to long range plans and policies and establishing project priorities for development of the most beneficial water project investment plan for the short term.

On April 3, 1984 His Excellency Minister Samaha issued Decree No. 63-1984 establishing a Planning Sector within the Ministry of Irrigation.

The new planning sector reports directly to the Minister and includes the following key Units, Resources and Water Uses, Planning and Followup of Investment Projects, Feasibility and Projects Evaluation, and Information Center.

This new Planning Sector will provide long range plans, investment plans, feasibility and preconstruction plans, provide followup on program implementation, and act as a technical secretariat to the Interministerial Water Planning Committee composed of members from various agencies involved in water resources. Other activities of this sector include providing an annual national water balance, maintaining a water resource data base, implementing the Nile River Irrigation Data Collection System, assisting in implementing computer models into actual irrigation system operation, and providing technical water resources information to other entities.

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1- Project Documents Water Master Plan for Egypt Phase I and II.

2- Master Water Plan Phase I final report (Main and 20 technical reports).

3- Egyptian Ministry of Irrigation Strategies for Ground Water Development and Re-Use of Drainage Water for Irrigation.

4- Permanent Joint Technical Committee for Nile Water reports on the development of Nile Waters for Sudan and Egypt.

5- National five-year plan documents.

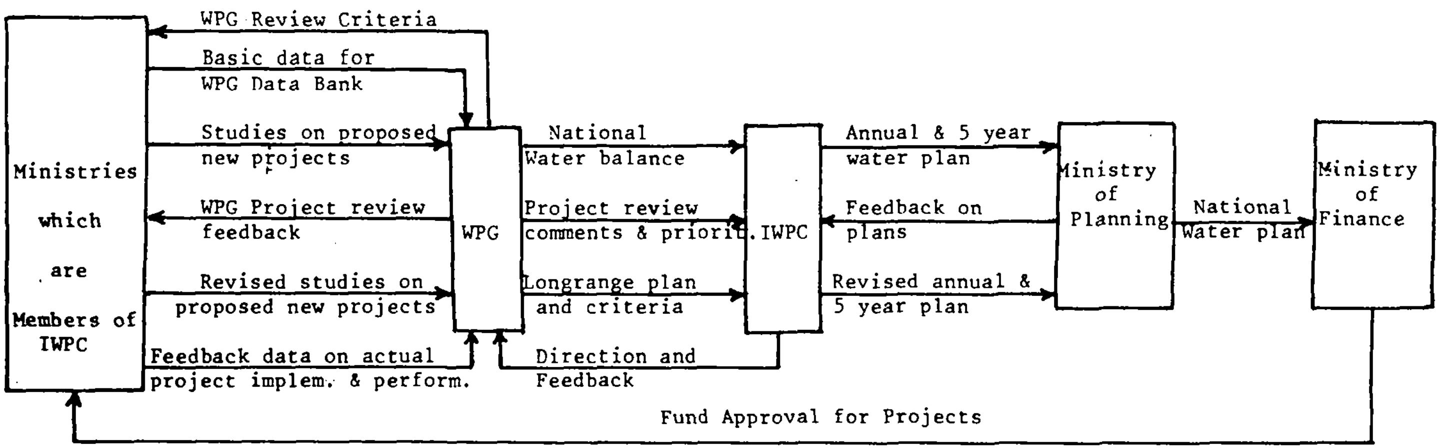
6- Ministerial decrees for the establishment of WPG and IWPC.

7- World Bank reports on the Agricultural Sector in Egypt.

CHART I

ANNEX I

IWPC WATER DECISION MAKING PROCESS



WPG is Technical Secretariat for IWPC
IWPC - Interministerial Water Planning Committee
WPG - Water Planning Group

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 196b Aspect number 1

WATER AND RURAL DEVELOPMENT WITH SPECIAL REFERENCE TO EGYPT

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ABSTRACT

The water resources represent the cornerstone for development in different fields of life. No doubt, water lies at the heart of rural development, which is the main motivating power for rural societies progress and in turn decrease the gap between rural and urban areas, thus it leads finally to the whole country development.

The presented subject discuss the main elements of rural development, its objectives and its role in the country development. Then the water resources development in the rural areas are discussed, its policies and the planning methodology for the maximum use of the drop of water in the fields of; irrigation, drinking water and municipal, industry, hydropower, navigation, tourism and recreation.

The Author discusses the planning process for different water projects and its role in rural communities development and in increasing the capabilities of the rural society and in decreasing the social stratification, and finally discusses the role of irrigation mechanization and water use rationalization in rural development.

The Author also discusses the legal and institutional aspects with regard to water use and water scheduling in the agricultural societies and how the law can be considered as the governing factor of the whole process of development, specially in the field of extension. Also the paper discuss the cooperation between the members of the rural society & between them and the executing agencies in the field of operation and maintenance for the different water structures.

Finally the Author discusses the Egyptian concept and the interaction between the river Nile -as the lonely source of water- and the Egyptians.

The paper introduces some recommendations for the future of water resources development in rural areas in the third world.

INTRODUCTION

Rural development is the motivating power for the overall development process since it develops one of the biggest society in the country and decreases the gap between rural and urban areas.

The rural development may be defined briefly as, "the integrated process of fundamental social and economical changes in agrarian societies that cover all sectors to reach a common set of development goals based on the capacities and needs of the people".

The process of rural development include many activities such as; agriculture, agroindustry, health care..etc., all of which depend mainly on the availability of a permanent source of fresh water which is the main foundation for rural development since the dawn of history. It is very clear that, there is always a common factor to be secured in the development process which is fresh water.

Since fresh water plays the main role in developing the rural society, therefore it will be our main concern to discuss the water potentialities in the development process, the suitable policies for each society and the planning methodology to develop and maximize the use of each drop of water taking into consideration the past and the future experience in that field focusing on the Egyptian concept as one of the oldest civilization affected by the availability of water resources.

FIELDS OF RURAL DEVELOPMENT

The rural development is proved generally by developing many fields in the rural society, where the main ones are; natural resources, agriculture and agroindustry, health care, nutrition and fertility, municipality and sanitation, power generation, education and training, infrastructure and rural urban links, and finally institutional and policy process.

No doubt each of the prementioned fields is very important in developing the society, but we can consider the development of natural resources and agriculture as the main issue since the other activities are depending mainly on those two fields.

WATER IS THE FOUNDATION OF RURAL DEVELOPMENT

Back to the history, where the old civilization started along the rivers or at the oases when a permanent water supply was secured since water is the main element for human consumption, sanitation, agriculture, production of industrial goods, navigation, power generation, transport and recreation. Thus the water planning and methodology of water development have attracted the attention of the responsibles for water in the different societies, specially when considering that the misunderstanding of water nature may cause disaster for the society. Therefore it was well understood since thousands of years that dealing with water requires skilled planning and careful management. This concept will be stressed when considering the Egyptian experience across the history, where the interaction between man and water have been developed in many ways and by different means. The main philosophy was always to fulfill the society requirements and prevent harmful effects. The water projects were matching with the different needs in the frame of the simple technology acquired. The water planning methodology was characterized in each era by a main criteria and in the meantime, it was focusing in developing a certain field of development activities untill we reached the multipurpose concept of today where the drop of water has the potential to cover many fields without dangering the environment within the society.

WATER POLICIES

The water policy generally emerges from thoughts and concepts that crystallizes into actual implementation programs for proving the rational use of water resources available in the society.

Back to the history we notice that the water policies were very simple and based on a linear relationship between the society needs for a short period (less than one year) and at a specified place and the available water resources. Later on the philosophy changed since many activities were considered at the same time and the policy was drawn up to elapse for a number of years. The water policies nowadays became very complicated and requires the adaptation of mathematical models and programmes to solve the complicated relation between the available water resources and the multiuse concept considering the following main points:

1- The economical, social and political evaluation are the governing factors for the policy validity.
2- The society circumstances should be thoroughly considered.
3- The policy span should be generally between 10 and 20 years.
4- Legal and administrative arrangements are the main boundaries.
5- The policy should be dynamic to permit for any future changes.

PLANNING METHODOLOGY

The term plan may be defined as a program which lays down the means, tasks and times for achieving a given result. With regard to water planning across the history, the main concern of the planners was to develop the society and secure its requirements. Thus the planning methodology passed in a series of stages summarized in the following points:

1- Trial and error to gain a certain target mainly in the field of securing water for the society needs.
2- Programs for implementing a project to reach a certain result; no scientific planning measures were considered.
3- Planned programs for achieving specified goals within a fixed time span in a certain area.

4- Comprehensive plans for the multiuse concept of water resources.
5- Using modelling technique and computer facilities to draw up master plans for water resources development and best use.
6- Drawing master plans for the International water system.

Considering the Egyptian concept, we find that the Egyptians experienced the different stages of planning methodology while considering the following boundaries:

a- They have only one source of fresh water which is the Nile.
b- The seasonal variations and uneven distribution of the Nile.
c- The multipurpose concept, such that the investment for one item can always be shared by other items.
d- Eight African countries beside Egypt are sharing the Nile.
e- The topographic & area of agricultural land are limiting factors for the development process.

Stemming from these boundaries the Egyptians started systematically to prove their march for acquiring the main objectives in the planning process.

First Era (4000 B.C. 1850)

The Egyptian's main concern during this era was to save their civilization against flood hazard and develop their society through agricultural extension, beside using the Nile as a navigable water way. They controlled the Nile between two strong banks, built a dam at Memphis across the Nile and a seasonal storage reservoir at Mories lake in Fayoum. They also executed some very deep canals to secure irrigation and experienced different water lefting devices as Saquia, Shadouf, Tampour, Zawafa, etc. They did not concentrate only on the Nile but they built dams to store the torrential flows as Sudd El-Kafara dam to secure drinking water for labourers in the alabaster quarries near Helwan. They also developed the groundwater reservoir specially at Sewa oases in the western desert and in Sainai. All these works were the foundation to extend and develop the rural societies allover the country.

Second Era (1850-1964)

The main targets during this era were to maximize the summer water, secure navigation all the year and secure power generation from water falls to develop the old societies and establish new ones.

This era started mainly when Mohamed Ali the Ottoman Governor tried to put Egypt economy on a modern footing. His point of attack was the agriculture development through water resources development and best use. Barrages across the Nile were built and hundreds of canals were constructed and the agricultural land reached 6 million acres out of which 1 million was under basin irrigation. New agricultural societies were established and the economy as a whole started to flourish based on the agrarian society development specially when electricity was generated from old Aswan Dam.

Third Era (1964-1977)

This era started with the construction of the High Aswan Dam since the 6 million acres became under perrenial irrigation and new one million acres were added to the agricultural land and 2100 migawatt of electrical power were generated to contribute to about 70% of the total power consumption in Egypt at that time. The main features of that era were:

- Safeguarding the country against high floods and developing the rural society through extensive agriculture and electrification.
- Starting an agroindustrial development programmes in the rural societies.
- Flexibility of agricultural planning & crop pattern.
- Possibility of drawing up a master plan for water development and best use, and in the meantime develop navigation & power generation.

Fourth Era (1977-2000)

The Egyptians are trying in this era to maximize the use of each drop of water and develop their rural societies through adapting suitable technology and/or generating national ones. Two main components are considered to prove the suggested goals.

Drawing a Master Plan for Water Resources Development

The planning sequences drawn by the Egyptians are:

1- Establish overall growth criteria for water planning purposes.
2- Set general and specific goals for water planning.
3- Establish composite demand scenarios from item 2 & rank water supply options.
4- Match water supply and demand considering other component of growth scenarios not served by the main water resource (the Nile).
5- Assemble plan components from the foregoing, evaluate the plan and compare plan evaluation.

Irrigation Development and Water Management

The main goal in this respect is to develop and readjust the water structures and irrigation systems to match with the new concept of rational water use. The main outputs expected are as follows:

a- Increase of agricultural land, agricultural production, and national income.
b- Raise the capabilities of labour force in using new technologies.
c- Save water and raise the water use overall efficiency.
d- Facilitate the farming procedures and farm mechanization.
e- Increase the social cooperation and decrease social stratification.
f- Decrease the water born diseases and increase health care.

WATER DEVELOPMENT ROLE IN SOCIAL STRUCTURE

The interaction between man, society development and water availability is characterizing our march to reach better life. On one hand, social structure determine water use and water demands while on the other hand water use availability and demands will affect the social structure. Therefore we should look to water resources both as an opportunity and agent for social changes and as a constraint on social structure and behaviour of both the people and the government.

Considering the Egyptian concept across the history, we find this interaction clear specially in the following fields.

Political Aspects.

Egypt is depending mainly on one water source which is the Nile. Thus, the Nile valley was one of the first world centrally managed hydraulic system, where the people hoped always of a central strong government to notify them with full information about the Nile water and its flood. Every body was self convinced to accept and obey any orders to ensure safety.

The Pharaos controlled the whole country through one strong government and then divided the country into governorates matching with the boundaries of the main canals. They constructed a powerful structure for supervising the waterways where the irrigation engineers were having the power to enforce all civilians to work for saving the country against floods, obeying rules of water distribution and keeping waterways in a good case. The Nile valley was always a peaceful land concentrating on defence aspects and efficient communications between the different villages to cooperate quickly in case of danger. The year in Egypt was divided to 3 seasons matching with the Nile regime (Inundation, coming forth and lack of water). The taxes paid by the citizens were in the form of forced labour (Corvee) to clean waterways and maintain different structures, but such concept was completely changed after covering the country with a complete irrigation network and the civilians started to concentrate their effort in the developing process and pay direct taxes.

Social Aspects

The social structure and social activities is affected completely by the availability of water resources, policy of land ownership depends mainly on the available water and the methodology of its distribution among different beneficiaries. The water development is generally affecting the society in the following:

1. New agrarian societies will decrease the dependance on the big towns, decrease immigration to it and give more weight to agrarian issues. This will make the rural society as the driving force for the entire development process.
2. Decrease the gap between rural and urban areas, prove social integration and decrease social stratification.

3. Prove the flexibility of agricultural planning, reduce poverty, develop labour force and improve the environment.
4. Modernize the agrarian society and raise the farmers capabilities and in turn develop the social structure.

The prementioned activities were proved in Egypt through the Egyptians march to develop their water resources. The land was completely owned by the government, then changed slightly according to the availability of water. In 1855 only $\frac{1}{7}$ the irrigated area was privately owned. By 1896 after constructing barrages & canals the bulk of the irrigated land was privately owned. By 1908 the whole irrigated land was under private ownership due to the extensive network of waterways and water structures. The forced labour decreased gradually untill it vanished completely. Many agrarian societies were established following the development of irrigation works. The link between different societies was proved through the waterways network and affected the social structure allover the country. Health care due to water availability raised the efficiency of the whole society. The new module of water development affected the citizens capabilities and improved their habits and method of thinking as well as improving cooperation among the different families and societies.

ROLE OF IRRIGATION MECHANIZATION

The irrigation mechanization potentialities in rural societies are the base for new development process since it can prove the following objectives:

1. Increase the efficiency of the conveyance system, prove water management and develop the environment.
2. Develop the utilization mechanization (water supply, irrigation, industrial uses and energy production), raise the farmers capabilities through their experience of the adapted technology, increase social cooperation and decrease social stratification.
3. Increase the potentials of agricultural mechanization, facilitate farming procedures, saving animal power, decrease the labour force required and prove the agroindustry.
4. Save water, prevent water logging and salinity, raise the efficiency of drainage network and control aquatic weeds.
5. Decrease the water born diseases and save millions of peoples from serious diseases, thus increasing their productivity.
6. Decrease the number of field laterals and in turn increase the agriculture land.
7. Keep land fertility, increase the society production and in turn develop the national economy.

LEGAL AND INSTITUTIONAL ASPECTS

The efficient mechanism of legal and institutional arrangements represent the cornerstone for water and rural development. We are trying here to highlight only the main philosophy in both fields without going in any details.

Legal Aspects

The main philosophy of water resources legislation in rural societies is to regulate the water development and use on the basis of justice and equity in the frame of the country wide benefit. Generally the water law is focusing mainly on the following:

1. To regulate the water ownership and/or the right to use it and to insure the protection of different water structures as a public utility.
2. To emphasize the standards of water quality and prevent its contamination.
3. To determine the boundaries of the governmental ownership related to water structures and to determine the role of regional and local levels of the government holding responsibilities for water control, distribution operation, maintenance & issuing different licences for using the public or private waterways.
4. To regulate the use of private waterways and structures without causing any harmful effect for others.

Institutional Aspects

The strong well managed institutional mechanism is the cornerstone for proving the society goals in water and rural development. The institutional philosophy may include the following:

1. To establish a central mechanism for coordinating all water interests at the national level and drawing the broad policy measures.
2. To nominate a body to have the responsibility for preparation, implementation and evaluation of plans and policies for the development and management of water resources within the frame of the National development planning.
3. To nominate an efficient mechanism for, training, education extension, operation and maintenance in the fields of water resources development and best use.
4. To establish an appropriate organization to draw up short medium and long term plans for revising legal and institutional aspects to match always with the society structure and requirements.

THE RECOMMENDED MODULE FOR DEVELOPING COUNTRIES

The nowadays crisis in the third world is concentrated in three main aspects:

1. To manage and develop the natural resources and rationalize their use.
2. To develop and extend the agrarian societies in order to prove food security and self sufficiency.
3. To reinforce the National mechanism in the fields of education, training and extension and to concentrate in generating National technology in the fields of water resources and rural development.

Based on the Egyptian experience the following module could be adopted to prove water resources and rural development in the developing countries.

1. Determining the elements, factors and potentialities of rural development, and to outline the main goals of the development process and prepare for data collection, processing & monitoring.
2. To draw up appropriate policies for natural resources development and establish project priorities that will affect the best economic returns from the utilization of natural resources considering the following points:

   a. To balance between moving people to water or moving water to the people and to draw the policy of private versus public ownership of developing tools and resources.
   b. To study thoroughly the relationships among water needs, water demands, population density and socioeconomic development.
   c. To determine the constraints and problems hindering the past development and draw the procedures for overcoming them.

3. To establish the rules for the best functioning of water establishments for their optimum exploitation and draw up priorities for the different uses of water focusing on:

   a. Distribution of benefits among users and to study how the engineering services are provided to and distributed throughout the society and its effect on various classes.
   b. To focus on managing and developing the social system.
   c. To consider the pilot projects as a tool for the development process.

4. To adopt mathematical models and programmes liable to future modification in order to realize the best use of water resources.
5. To study the availability of energy and the procedures of its development.
6. To lay out the developing plans and to evaluate the effect of each activity on the development process.
7. Determining the implementation programmes, financial & technical requirements, procedures of follow up and main outputs focusing on agriculture and agroindustry as a motivating power for the development process.

CONCLUSIONS

The water resources development and rational use is the main tool for the rural society progress and in turn represents the motivating power for the overall development process specially in the developing countries. Therefore, the development process must consider the interaction among man, water and environment and to prove the political, social and economical equilibrium in the society. Such criteria requires a thorough knowledge of the problems and constraints hindering the past and/or the present development focusing on inherited habits and religious beliefs as well as the social structure.

The planners may be able to have a future expectation for the effect of the development process on the different classes of the society and for proving their goals in the society development. The development process in the field of water resources may consider the following main points:

1. The water policy should be dynamic to permit for future changes.
2. Moving water to the people or moving the people to water is to be weighed in the frame of political, economical & social equilibrium.
3. Legal and institutional arrangements are governing factors for proving the development process.
4. To prove the continuity of the development process specially in the fields of training, education and extension.
5. The boundaries between public and private ownership of the developing tools must be very clear.
6. To ensure the continuity of evaluation and follow up of the different programs.

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IRRIGATION DEVELOPMENT IN RURAL SOCIETIES OF EGYPT

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ABSTRACT

Irrigation development is one of the main foundations of rural development, since it leads to the best utilization of all available water resources, protects the land fertility and results in the overall development. It also has a positive impact on drainage efficiency, aquatic weeds control, public health, individual and collective capacities to learn new technologies, social cooperation and the national income.

The planning for irrigation development in rural societies is greatly associated with the capacity to get optimal benefit from existing structures and developing them in the best economical way. It may depend mainly on modernization of the existing facilities and tools using simple technological means and also to change the related traditions and wrong religious beliefs.

This paper presents the Egyptian experience gained through applied research and field work. The general objective was to improve the social and economic conditions of small farmers through development and use of improved irrigation water management and associated practices which increase agricultural production, promote efficient water use and reduce drainage problems. The main development programmes are as follows:

1. Improve management and distribution of irrigation water.
2. Minimize transmission, distribution, operational and application losses.
3. Improve farm irrigation systems and land levelling.
4. Help the farmers to form water users associations.
5. Train the farmers on the proper water use and improve water lifting devices.

The approach taken to implement such programs is to follow a systematic procedure comprising, problem identification, search for solutions, pilot projects and developing procedures for disseminating practices which were proved through the pilot projects. This work demonstrated the value of an interdisciplinary approach and national improvement programs outlined on this basis.

INTRODUCTION

Irrigated agriculture represents the backbone of the rural development since it proves the national wide goals in the different activities related to agriculture such as, food production, agroindustry, agricultural mechanization, labour force, social cooperation etc..

Generally irrigation has provided new opportunities for more intensive crop production, but it has also generated new problems for, the soil, water conservation, drainage system, health care and crops productivity. The resulted problems are mainly due to the low efficiency of the existing systems and/or the misuse of the available water resources either by the individual farmers or by the society as a whole. The challenge, then, is to minimize or solve these problems while fully exploiting the new opportunities for the benefit of the nation, i.e., to maximize the efficiency of the existing systems and to suggest the solutions for the problems hindering the past development rather than thinking in new systems which will require huge capital investment.

The irrigation development concept is mainly to manage and regulate the use of the development tools in that field such as; water resources, irrigation facilities, drainage systems, soil, cropping systems, law, institutions, farmers and procedures in order to maximize the use of each drop of water in the frame of justice and equity and to minimize the side effects to their lower limit.

The importance of the suggested development of the irrigation systems is resulting from the fact that, an estimated 250 million acres of irrigated land are spreading allover the world. The rate of increase of the irrigated land is about 15 million acres yearly, while the water resources quantity is limited. The average allover efficiency of water use in irrigation ranges between 45% and 55% which is very low, such criteria calls for comprehensive programmes for developing the irrigation systems and rationalizing water use which will finally result into the following:

1. Conserve water supplies for extending the irrigated areas and for rapid increases in food production.
2. Improve the returns on investments of existing systems.
3. Reduce the costly water logging and salinity problems which are often due to poor management.
4. Reduce the need for large capital investments in new systems.
5. Gain knowledge which can provide new criteria for the development of other areas.
6. To help the small farmers in solving their problems and to improve their standards of living.
7. Increase the national production through developing the rural society.

It is worthmentioning that the irrigated 6 million acres in Egypt acquire comprehensive programmes for irrigation development and water management to overcome the serious problems accumulated since many years. The Nile Valley and Delta lands have been under continuous cultivation for thousands of years. The Egyptian lands started to suffer from low productivity mainly due to mismanagement of water use and low efficiency of irrigation systems. The irrigation practices which used to be adequate in the past have now become impractical to solve the pressing need of more production to ensure food security, specially when considering water as the limiting factor for agriculture extension.

Egyptians started since a number of years to practise the irrigation development and water management through a transfer process which resulted in rapid adaption of appropriate technologies in these fields. In the meantime they tried to generate national technology to match with the social structure in the rural areas. The positive results gained is considered as a base of the national irrigation development programmes which started in 1984 in an area of 125 thousand acres as a first step. The national irrigation development plans which was approved by the cabinet in 1981 calls for developing the irrigation network and structures as well as to prove water management concept allover the irrigated area in the country.

IRRIGATION DEVELOPMENT POTENTIAL

The irrigation development is the key stone for agrarian society development and in turn will increase the society production and lead to prosperity and welfare. The development process depends on selecting the right package of suitable technology to exploit the different potentials of the whole process. Irrigation development and water management will lead to the best utilization of all available water and land resources and increase the national economy. It has also a positive impact on the people, society, and the government.

Optimal and Rational Utilization of Land and Water Resources.

Irrigation development and water management programmes are intended to ensure the most efficient use possible of the available water resources and will also lead to the improvement of the soil fertility and intensive production. The main outputs of the development process can be summarized as follows:

1. Increase efficient use of water resources by controlling and developing both the transmission and application systems and by limiting the flow to the actual requirements.
2. Minimizing the water losses to its lower limit by applying an efficient distribution and monitoring system.
3. Limit the flow to the drainage system, reduce the drainage duty and insure better drainage conditions. This will lead also to keeping soil fertility and increasing its productivity.
4. Control the water use on the farm level and alleviate water logging and soil salinity problems which occur due to excessive seepage either from the system or from the irrigation water.
5. Conjunctive use of surface and groundwater will lower the subsoil water level and increase land productivity.

6. Increase crop productions through a developed on farm water management programmes.
7. Help in controlling aquatic and other weeds.

Improvement of Socioeconomical and Cultural Conditions.

The irrigation development is not only a process for developing the existing irrigation and drainage network, but it will have a positive impact on many other fields such as; mode and way of life, labour force, social set up, farmers' standard of living, way of thinking, inherited habits and religious beliefs. The main features of the socioeconomical and cultural development are briefed in the following main points:

1. Overcome the problems related to water born diseases, improve the public health and increase the productive capacity of the agrarian society.
2. Increase individual and collective capacities to learn new technologies and its adoption to confront with rural environment.
3. Increase the capacity of rural community to work as homogeneous teams specially through the water users associations.
4. Increase the social cooperation and decrease social stratification.
5. Increase the potentials of agricultural mechanization, facilitate farming procedures, saving animal power and prove agroindustry.
6. Improve the inherited habits and traditions and correct the wrong beliefs.
7. Increase the total income for both the individuals and the nation.
8. Introduce an efficient base for legal and institutional disciplines in the society and prove self dependance in the rural areas.
9. Fulfill the national wide goals in developing the nation with the least investment required.

THE NEED AND STEPS FOR IMPROVEMENT.

The land area to grow irrigated agriculture has been extended & developed to the limits of water resources, soil conditions, existing facilities and tools, legal and institutional arrangements. Therefore, there will be an urgent need for a comprehensive development process to maximize the use of available water and land resources in the frame of efficient institutional and legal arrangements. The main steps for improvement are briefed as follows:

1. Increase the effectiveness of the development process by improving the systems and procedures of water distribution.
2. Conserve the conditions of water source and irrigation system facilities as a whole.
3. Improve the skill and ability of irrigation personnel at all levels to carry out the development programs.
4. Support the research and study of soil-water and plant relationships.
5. Establish improved water control practices for the water delivery and distribution methods.
6. Develop plans for organization and implementation of

expanded future programs based on results of the pilot projects.

7. Develop and train qualified engineers and technicians who carry on and conduct on farm water management and other development activities.
8. Arrange for the legal and institutional measures of the development process.
9. Draw comprehensive plans for efficient module for operation and maintenance to insure better functioning of the whole system.

IRRIGATION DEVELOPMENT PROGRAMMES

Irrigation development programmes are serieses of several parallel and/or overlapping phases which require skilled planning and careful management since it involves many measures to be taken in order to satisfy the suggested goals in the frame of the society requirements, traditions and habits, religious beliefs. and the optimal use of the existing structures. The suggested programmes would match with the following main points:

1. The country's development objectives and programmes.
2. Technically sound and the best of the available alternative under technical and other constraints.
3. Administratively and legally workable.
4. Economically and financially viable.
5. The adapted technology must be redeveloped to be in conformity with the socio-culture and economical conditions of the society.
6. The end choice for improvement type will depend on the cost benefit and the type of problems constraints in order to reach optimization of water and land resources.

Generally, the development programmes are mainly to permit the traditional system to appropriately and systematically impact the countries research and development efforts. The most important phases of the development process can be grouped in four phases.

Priority Problems Identification.

The unique features of problem identification is the interdisciplinary approach with farmer participation to achieve well understanding of system operation. This will lead to a thorough definition of priority problems. The main philosophy of this phase is to determine the main problems hindering the past development and the limiting boundaries in order to determine the highly visible solutions since all problems cannot be solved initially.

Search for Problem Solutions.

The problem solutions phase is expected to produce results highly visible to the farmers and to the national economy. Direct solutions from past experience known principles are solved first and adopted to specific problems and resources. Demonstration of suitable known technology is the next priority. Applied research is reserved for evaluating complex alternative solutions to high priority problems which require more careful study.

Assessment of Solutions.

A systematic quantitative assessment of each solution is made to ensure society acceptance and evaluate socioeconomic, cultural and environmental impacts. The applied, adaptive and evaluation research methods under the existing conditions are tools for the assessment of the solutions. The assessment process is used to determine resources, communications, legal and institutional needs to ensure the continuity of the development process. The suggested solutions are to be evaluated overtime to select the suitable technology package for the pilot projects. Socioeconomic and cultural environmental impacts are also monitored for long term projections.

Pilot Project Implementations.

The pilot projects are considered as the main tools in the development process. Therefore, it is of great importance to design, implement and manage the pilot projects very carefully since it will be the base for the wide national development. Although the technology known can be adopted easily but a major constraint will be the lack of effective transfer of the acquired technology to match with the social structure. We should focus then on the development of selected institutional capabilities for effective transfer of technology by carefully designed training and evaluating strategies. First of all when we start the pilot project, the institutional capabilities and appropriate technologies are evolved in effective development programmes. The final step will be to evaluate the main outputs and weigh them in the frame of the social, economical and environmental boundaries. Then to select the suitable solutions which will be the base of the national development programmes.

IRRIGATION DEVELOPMENT IN EGYPT

Irrigation is the backbone of Egypt's agriculture, where the Nile represent the lonely source of water. The irrigation system and irrigation methods which the Egyptians practices are unique. They have experienced such methods since thousands of years. The irrigation systems which were constructed since many years started to deteriorate where the overall efficiency of water use ranges between 45% and 50%. Therefore, the full potential of irrigated agriculture can be realized with a significant improvement and modernization of the irrigation system and on-farm water management.

The irrigated area in Egypt is 6 million acres with 200% crop intensity. The water supply for any area is monitored by observing water surface levels in delivery canals. The water is typically delivered from 50 to 75 cm. below the ground surface of the fields so irrigators must lift the water onto the land. Farmers are not required to pay for water. The water use along the mesqa (private canal) is determined by customs which usually favours the farmers at the head of the mesqa. Similarly mesqas at the head of a distributing canal have an advantage over those at the tail end. The farmers distribute the water through field ditches to small bunded units called basins. Excess surface water is characterizing the Egyptian irrigation methods in addition to other problems which hindered the past development. The working on various programs to improve the irrigation system and prove proper management is the main outlet to the Egyptian march for overall development.

The Egyptian Government started in 1977 to draw an irrigation development strategy up to year 2000 which comprised three main stages:

1. Control and regulate water transmission and distribution.
2. Developing and raising the on-farm water efficiency.
3. Legislation and regulating the right to use water.

The first step in fulfilling the strategy was to identify the main problems related to the development process, where a technical, social, ecological and economical survey was conducted in the presence of the farmers. This procedure elapsed for about 2 years in selected areas representing the whole country. The main problems are as follows:

1. Low efficiency of the irrigation tools and measures either on the public or the private levels.
2. Lack of regulating and measuring equipments.
3. Inadequate operation and maintenance.
4. Tail reaches do not get enough water.
5. High water table and soil salination due to seepage and absence of conjunctive use of surface and groundwater.
6. Low efficiency on the farm level due to, inadequate land levelling, traditional irrigation methods, and poor design of on-farm irrigation system.
7. Limited access to fields for machines retards modernization.
8. Poor extension services, lack of pilot projects and demonstration farms.
9. Insufficient coordination among irrigation distribution, irrigation application and extension.

The second stage was to determine and establish the use of optimal irrigation systems, modernize and develop the irrigation network and structures and develop on farm water management specially in representative pilot areas. This stage include also developing plans for organization and implementation of expanded future programs based on results obtained from pilot projects.

PILOT PROJECTS

The Egyptian government started with United States Agency for International Development in 1977 comprehensive cooperational programs and pilot projects for fulfilling the Egyptian goals in the development process. Two main projects started, irrigation management system and on farm water management.

<u>Irrigation Management System (IMS)</u>

The IMS was established in 1980 in order to update and improve the functioning, operation and maintenance of the present irrigation and drainage systems and to prove its performance to the designed potential. The project objectives were:

1. Realization of the maximum long term benefits from the project.
2. Upgrading and improving the existing irrigation systems.
3. To have full control on water regulation, transmission and distribution.
4. To manage the water delivery and prove conjunctive use of surface and groundwater.
5. Reinforce training, education and extension organizations.
6. Establish an efficient institutional and legal measures.

The IMS project started and have been completed in 5 governorates where it fulfilled its main objectives. The other 20 Governorates will be covered in the next 10 years.

<u>Egypt Water Use And Management Project (EWUP)</u>

The EWUP started in 1978 in 3 pilot areas representing the different areas of Egypt with respect to soil, climate, traditions and crop pattern. The main objectives of the project are to identify the major constraints, establish the use of optimal irrigation practices and application methods, to develop plans for future programs based on the optimal results and to train qualified personnel who carry on and conduct on farm water management activities. In each of the pilot areas, project activities are implemented in 3 overlapping and interrelated components to prove the required objectives.

THE NATIONAL IRRIGATION IMPROVEMENT PROGRAM (NIIP)

The NIIP was designed as a result of the outputs of both IMS and EWUP. The project was approved by the cabinet in 1982 where the long term objective is to modernize the irrigation system and prove on-farm water management. The first phase is the execution of a package of interventions in an area of 125 thousand acres, in the areas where the pilot projects were implemented.

<u>Overview And General Description of The Project</u>

Main, branch canals, meskas, drains and field access systems will be improved so that farmers can maintain moisture conditions which are optimal for crop production and soil conservation while at the same time saving water for other uses. The irrigation and field access system will be improved to serve the future not the past. When possible, canals and meskas will be of improved earth design. Some may require concrete lining. A computer assisted design and evaluation procedure, developed through EWUP, is available to help adapt specific improvement alternatives to the local conditions.

On-farm irrigation systems can be improved only when main & branch canals and meskas are improved so as to provide a reliable source of water for each farmer.

The Ministry of Irrigation has decided to modernize the irrigation delivery system to accomplish the following goals:

1. Provide for more efficient water use, and increase crop yield.
2. Use water saved to irrigate additional lands.

NIIP is planned so that the MOI will implement the proposed regional improvement program while the private sector will be encouraged to provide contract services for constructing and maintaining the irrigation system.

Project Costs And Time Schedule:

The total project cost is estimated to be approximately L.E. 65.450 million. The time schedule is estimated to be about 5 years. This represent the first phase for an area of 125 thousand acres.

Project Output And Benefits:

1. Construction of improved facilities for equitable delivery of irrigation water to the farms with reduced waste of water and increased energy efficiency.
2. Delivery of irrigation water scheduled to meet crop consumptive use requirements.
3. A saving of at least 10% of water currently used and approximately 25% increase in crop productivity are expected.

Organizational Arrangements:

Design, implementation and supervision of the program will be the responsibility of the Irrigation Department. Technical and research requirements to implement the program will be under the guidance of the specialized water research institutes. The establishment of a new specialized authority to carry out all future national programs will be considered later according to experiences gained in the first 5 years plan.

CONCLUSIONS

The increased population allover the world calls for urgent development programs in the fields of agriculture, agroindustry and agrarian society development. The main key for the development process is the development of irrigation facilities, institutions, legal arrangements and on-farm water management.

The estimated ultimate irrigation potential is based upon present stage of development of suitable technology to match with the socioeconomic and ecological conditions in the society in the fields of water resources development and best use. Based on the Egyptian experience in that field the main target for the improvement process should be put at an achievable level according to individual and society conditions. The main foundation to prove this concept is to establish an efficient mechanism for technology transfer and/or to generate national technology for achieving the main goals. Training, education and extension are essential elements of the development process and should be coordinated and supervised on the different levels by an efficient mechanism. It has been found that the improvement of irrigation facilitics and on farm water management should cover the following:

1. Establish simple and efficient procedures for the development of the existing irrigation facilities.
2. On-farm water management programs should cover all the activities related to obtaining the maximum water efficiency and to raise crop productivity.
3. Activities and performance of all staff of the different organizations related to the development process should be controlled and regularly monitored.
4. Insure closer coordination among the agencies involved in the development process.
5. Operation and maintenance should be an efficient continuous process.
6. Water user's organizations should be established and encouraged.

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A SMALL-SCALE HYDROELECTRIC POWER DEVELOPMENT IN RURAL AREAS IN RECENT JAPAN

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ABSTRACT

After the oil shock in 1973, hydroelectric power development which was rather in decline has been evaluated again, because of the rise of petroleum price for thermal plant. This paper presents the role of hydroelectric power development after the oil shock, reviewing the historical background in modernizing Japan since the end of last century.

1. Introduction

After the oil shock in 1973, hydroelectric power development which was rather in decline has been evaluated again, because of the rise of petroleum price for thermal plant. This paper presents the role of hydroelectric power development after the oil shock, reviewing the historical background in modernizing Japan since the end of last century.

2. Historical review of small-scale hydropower development in Japan

In 1890, the first hydroelectric power plant in Japan was constructed in the Ashio Copper Mine ten years after the world first plant at Appleton in the United States. This first plant was for public purpose, with 750 kw of capacity and began its operation with the completion of the multi-purpose canal project, linking Lake Biwa to Kyoto, in 1892. This project planned in 1883 by Sakuro TANABE, as his graduation thesis at Engineering College, the old faculty of Engineering of the University of Tokyo, received the Telford Prize in 1895. In 1895, the first municipal electric tram in Japan was opened in Kyoto, utilizing its energy from this project.

At the beginning of the 20th century when the Second Industrial Revolution was in progress in Europe, electric power engineering developed rapidly with the progress of the First Industrial Revolution in Japan, as the steam-power and gas-light had not yet developed in Japan.

In 1908, 15,000 kw hydroelectric power plant was constructed at Komabashi, and in 1912, 37,500 kw plant was constructed at Lake Inawashiro and long-distance transmission of electricity started. These run-of-river hydroelectric plants were already of world-wide scale capacity at that time, but all equipments were imported from Germany, the Great Britain, and the United States. The run-of-river plants were developed rapidly during and after the First World War in Japan, because of the war economical boom. Also, owing to abundant precipitation everywhere and volcanic geological formations in many upper basins, the stream-flows are relatively rich in discharge through the year, which is favorable for run-of-river hydropower systems.

The first reservoir-type hydropower plant was constructed in the River Uji near Kyoto in 1924, and next reservoir-type hydropower plant with rather big scale (Oi Dam with 53.4m in height) began its operation in the same year in the River Kiso with 45,000 kw of capacity. Furthermore, multi-purpose reservoir plannings including hydropower initiated in several rivers in 1939, but almost all these plannings stopped during the Second World War. After 1950, multi-purpose river basin development projects became vigorous in many river basins. Especially in the 1950's and the first half of the 1960's, the construction of big dams boomed for hydroelectric power and multi-purpose in order to advance the high economic growth of the 1960's. On the other hand, because of the energy revolution based on rapid progress of petroleum technology, thermal-power capacity with oil fuel became greater than hydropower capacity in 1962, after the predominance of hydropower over the past half-century.

Thus, the relation between hydro and thermal has been changing with time as shown in Table 1.

3. Present role of run-of-river hydroelectric power development

After the oil shock in 1973, the roles of hydropower and thermalpower have changed. That is, the run-of-river type hydropower developments were activated and are being considered again from various aspects as energy source structures. The aspects to be considered in studying energy alternatives are: the sudden rise of oil price for thermal plant which cut down the difference in price between thermal and hydro: inhabitants' opposition movement increased substantially especially for the construction of thermal and nuclear power plants owing to the environmental pollution etc.: the construction of reservoir-type plants is decreasing because there are few advantageous dam-sites left over after the construction boom of past years. There is difficulty to gain the consent of the local inhabitants: the rise of the construction costs including compensation.

Thus, after 1973 run-of-river hydropower development becomes important again as one of the alternative energies, though this energy had become economically disadvantageous after the energy revolution of the 1960's.

In Japan, small-scale hydropower system has been always important. As shown in Table 2, plants below 50,000 kw which are called "medium- and small-scale hydropower" actually constitute 95% in number of sites, and 60% in capability. Furthermore, there are about 400 plants below 1,000 kw, which are called "mini-hydropower". Therefore, the hydropower equipments in Japan range from large to mini-scale and are situated everywhere, being different from thermal- and nuclear-power systems.

The merits of small-scale hydroplants are as follows. First, they are domestic sources, so-called "clean" without pullution, and the supply of energy is continuous. The merit of domestic supply is especially valuable in Japan, because she must import almost 100% of the petroleum consumed. The research on hydropower potential.done by the government from 1956 to 1959 shows that of a total potential of 35 X 10^6 kw, 21.5 X 10^6 kw has already been developed. The remaining potential, that is, 13.5 X 10^6 kw is constituted by 37% of small-scale projects (less than 10,000 kw) as shown in Table 3.

To promote the development of medium- and small-scale hydropower projects, the items to be solved are as follows.

1. Administrative items

   How to increase in the standard of living of local people with this hydropower development, and how to develop these regions themselves with this development.

2. Economical and financial items
   - 2-1. Evaluation on life time of the project
   - 2-2. Economical effect of the project in regional affairs
   - 2-3. Rather high initial cost of the project
   - 2-4. Improvement of managerial measures
3. Technological items
   - 3-1. Standardization of equipment
   - 3-2. Construction work efficiency

Almost all the run-of-river hydropower plants have been constructed in rural areas, and also new various kinds of small-scale plants in rural areas are promoting after the oil shock, for example, utilizing irrigation canal, water-supply system, or independent power plant for dam operation etc. The electric power produced at these run-of-river plants by the small electric power company or farmers'cooperative association is often offered for sale to the big electric power company or electrochemical industry company etc.

Table 1 Hydro- and thermal-power capabilities and outputs

Year	Capability (MW) Hydro	Thermal	Nuclear	Output (MWh) Hydro	Thermal	Nuclear
1905	18	56	-	-	-	-
1910	113	145	-	-	-	-
1915	449	323	-	-	-	-
1920	825	553	-	3,166	649	-
1925	1,814	954	-	6,742	993	-
1930	2,948	1,552	-	13,431	2,342	-
1935	3,382	2,375	-	18,903	5,795	-
1940	5,127	3,946	-	24,233	10,333	-
1945	6,435	3,950	-	20,752	1,148	-
1950	6,763	4,008	-	37,783	8,482	-
1955	8,907	5,602	-	48,502	16,738	-
1960	12,678	10,979	-	58,481	57,017	-
1965	16,275	24,717	13	76,420	115,703	-
1970	19,994	46,932	1,336	80,089	274,868	4,581
1975	24,853	80,817	6,615	85,906	364,764	25,125
1980	29,776	98,234	15,689	92,092	402,838	82,591

Table 2 Distribution by scale of hydroelectric plants

Scale (kw)	No. of sites	Max. capacity (10^3 kw)
$10^3 >$	394 (24.1)*	160 (0.8)
$5\times10^3 > \quad \geq 10^3$	496 (30.4)	1,160 (6.0)
$10^4 > \quad \geq 5\times10^3$	262 (16.0)	1,760 (9.1)
$5\times10^4 > \quad \geq 10^4$	398 (24.4)	8,260 (42.6)
$10^5 > \quad \geq 5\times10^4$	61 (3.7)	4,020 (20.7)
$\geq 10^5$	22 (1.3)	4,050 (20.9)
Total	1,633(100.0)	19,400(100.0)

* figure in parentheses shows percentage

Table 3 Undeveloped medium-and small-scale Hydropower and Total Polential Hydropower

	Plants		Max. capacity		Yearly Output	
	number	percentage	10^3 kw	percentage	10^6 kwh	percentage
Medium- & small-scale (<10,000kw)	1,930	(51) 82	4,161	(7) 13	21,733	(15) 37
Large-scale (≧10,000kw)	410	(11) 18	27,357	(9) 87	37,422	(25) 63
Total	2,340	(62) 100	31,518	(56) 100	59,184	(40) 100

* figure in parentheses shows percentage

* figure under parentheses shows percentage to the total

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EXPERIMENTS BASED ON THE POTABILIZATION OF AUSTRALIAN SURFACE WATER BY MEANS OF DIRECT FILTRATION (OMNIFILTRATION$_R$) SYSTEM

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ABSTRACT

These experiments form part of a wider research trend which aims at ascertaining the feasibility of applying a system based on in-series direct filtration at high velocity, i.e., Ofsy, to treatments for the separation of high loads of undissolved substances or of substances likely to become undissolved. Throughout Australia, almost all surface water is characterized both by a highly coloured content and by a highly colloidal turbidity (the fineness coefficient or ratio between suspended solids and NTU turbidity is very low, i.e.: less than 1). A pilot plant was installed near the banks of the Murray river which provides a typical example of surface water in Southern Australia. The tests were run almost at the end of the dry season (March 1984). The information collected gave useful indications on the ideal operating conditions for carrying out water treatment and on further organoleptic and chemical parameters for determining the potability of water. This contributed to proving that direct filtration, Ofsy in particular, can therefore be applied to water having these peculiar characteristics. It could be observed that the colour is partially apparent because of the presence of algae such as Granulate Melosira and Brown Diatoms in summertime and to silt in wintertime, whereas the true colour is due to resins given out by the vegetationmainly formed by Stringy Bark Gum trees. These resins are highly soluble in water.

INTRODUCTION

The Ofsy system, based on in-series filtration, affords a higher capacity of retaining the dirt to be removed. This capacity is exerted above all by the first filtering unit whereas an efficient polishing action can be observed in the second filtering unit. In fact, owing to the diverse and peculiar chemico-physical characteristics of dirt, the second unit retains whatever has escaped the first filter (Coccagna, 1981). These are the reasons why Ofsy has already found its first experimental applications in the treatment of highly turbid water of fast flowing rivers; in the separation of algae from lake or reservoir water; in the clarification of and final phosphate removal from biological effluents; in the removal of high concentrations of iron and manganese from underground water. In all these cases, the use of coagulants like aluminium or iron salts serves several purposes as it stabilizes colloidal particles by providing counter-ions, it promotes flocculation and finally it chemically reacts to the formation of insoluble compounds. As to colour, aluminium salt as well as cation polymers have the capacity of forming insoluble compounds (Amy, 1981 - Hall, 1965 - Batchelor, 1982 - Mitchell & Hill, 1981 - Narkis & Rebhun, 1977 - Rest, 1982 - Chadik, 1983). Tests run in Norway with water characterized by humic and fulvic acids demonstrated how this capacity strictly depends on the pH (Coccagna, 1983).
Up to now, colour removal has been attained by employing conventional units whose performance is based on the coagulation-flocculation-sedimentation-slow filtration sequence. Direct filtration has been studied by some authors but it has never gone beyond the laboratory stage. Until recently, colour removal from potable water was considered as a mere organoleptic requirement. Only upon the demonstration that disinfection by chlorine and chlorine derivatives leads to the formation of organic halogenated compounds harmful to human beings, has this problem been regarded as a source of concern (Fleischacker, 1983 - Gaffney, 1977 - Reed, 1983 - Rook, 1974). Every expert agrees with the fact that the causes giving rise to this phenomenon ought to be removed in order to solve the problem. This would entail the removal of all those organic substances likely to react with chlorine (Coccagna, 1982 - Hoehn, 1980 - Olivier, 1980 - Vogt, 1981).
The best known substances among them all, the so-called precursors, are certain species of algae and above all colour (Anonimous, 1981 - Mc Creary, 1981 - Oliver, 1983, Qualls, 1983). This is the reason why the de-chlorination treatment is carried out either by avoiding disinfection and pre-chlorination or by injecting sodium hypochlorite at the inlet of the system. Particular attention was paid to aluminium leakage into product water as aluminium is highly harmful to patients undergoing hemodialysis treatment (Miller, 1984 - Davison, 1982). Upon this, the main target was that of attaining less than 5 C.U. (Pt Colourimetric Units) and less than 0.1 mg/l of residual aluminium.

MATERIALS AND METHODS

Pilot Plant

The pilot plant consisted of an Ofsy system. Each filter has a 20-inch diameter. Picture 1 shows the lay-out of the filtering bed. Picture 2 shows a diagram of the flow of the in-series filters.

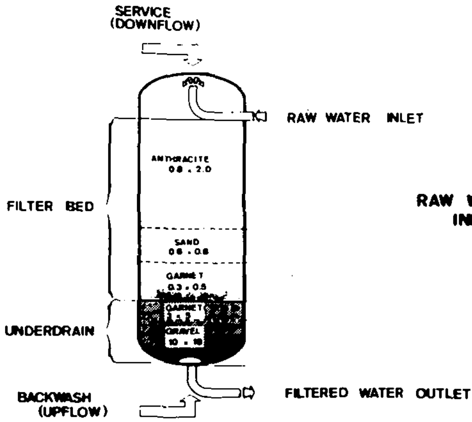


FIG 1

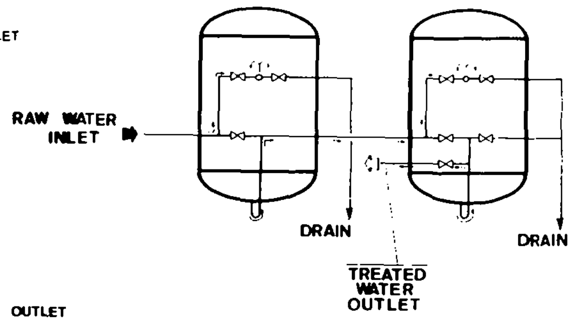
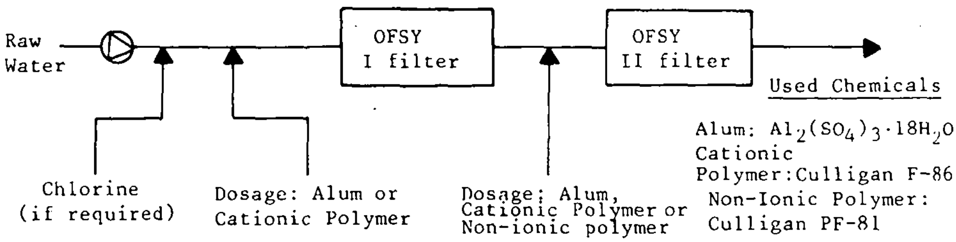


FIG 2

The following diagram shows the water treatment process (Picture 3)



The standard operating conditions of the Ofsy system are indicated in Table 1

Table 1 - STANDARD OPERATING CONDITIONS

Standard Conditions	Operation (Service)	Backwash
Water Flow	3 - 4 m^3/h approx.	7 - 8 m^3/h
Surface Load (velocity)	15-20 $m^3/m^2 \cdot h$	35-40 $m^3/m^2 \cdot h$
Chlorination (if needed)	2.5-3.0 mg/l (1.0-1.5 mg/l residual chlorine)	20-30 mg/l
Time	8-12 hours ·	20 minutes

Tests were run without any chlorine injection (hypochlorite sodium). Nevertheless a strong chlorination of the filters was carried out during backwash so as to prevent either microbic growths or any undesired clogging. Raw water was employed for backwash.

Water Characteristics

In March 1984, the pilot plant was installed near the Echuca town, 200 Km away from Melbourne - Victoria State. The water to be treated was drawn directly from the nearby Murray river.
The water analyses are set out in Table 2

Table 2

Parameter	Unit of Measuring	Given Values	Parameters	Unit of Measuring	Given Values
Apparent Colour	Hazen Units	46-50	Conductivity	μS/cm at 20°C	85-90
True Colour	Hazen Units	16-20	Total Hardness	mg/l $CaCO_3$	21
Turbidity	NTU	18-20	Iron	mg/l Fe	~ 1.5
Suspended Solids	mg/l	20-25	Manganese	mg/l Mn	0.06
pH	pH	7.4-7.5	Aluminium	mg/l Al	0.11
			Permanganate Value	mg/l O_2	5

Throughout the test period, 2 weeks approximately, the water characteristics remained almost unvaried. Only turbidity underwent slight variations. During each cycle, routine colour measurements were not carried out as no colorimeter was available on site. All water samples were sent to an equipped laboratory. It could, however, be seen that colour removal had a trend similar to that of turbidity.

RESULTS AND ENSUING DISCUSSION

By taking into account that direct filtration calls for a much lower consumption of reagents than that required by conventional treatments and the considerable influence of pH as demonstrated during several tests and as divulged in the literature, the preliminary tests run by the pilot plant mainly aimed at achieving an optimal consumption of reagents as well as an operating pH. Owing to the low saline content in the water and its almost nil tampering capacity, pH adjustment was not attained easily nor could be obtained an accurate value.
The results are indicated in Table 3, (turbidity meter HACH Model 2001 A).

Table 3 - PRELIMINARY TESTS

Influence of pH on Alum dosage

Alum Dosage mg/l	pH adjusted at 6 approx.		pH adjusted at 7.5 approx.	
	pH of treated water	NTU turbidity treated water	pH of treated water	NTU turbidity treated water
5	6.40	9.0	7.55	7.1
10	6.35	3.8	7.60	6.5
20	6.30	0.41	7.30	0.55
30	6.05	0.39	7.40	0.51
50	5.85	0.56	7.40	0.43
80	5.70	0.42	7.40	0.63

These tests contributed to demonstrating as follows:

- Even if pH was not easy to regulate nor accurate, it had no effects within quite a wide and significant range. The same applied for alkalinity. The peak pH values were achieved by injecting Na_2CO_3. It can therefore be said right away and rather unexpectedly that these two parameters have no part in colour removal.
- The alum injection (20 mg/l) necessary to obtain the required clarification of water appears to depend on an approximate stoichiometric relationship. Dosage excesses when compared with threshold values have no effects.
- The optimal value for reagent (about 50 mg/l) was discovered when running jar-tests which aimed at singling out the ideal dosage of Alum according to the conventional treatment based on flocculation-sedimentation. The advantage afforded by direct filtration was thus re-confirmed. Upon verifying the lack of pH influence, cation polymer was tested out instead of alum. The results are as indicated in Table 4.

Table 4 - Influence of Polymer Dosage

Cationic Polymer Dosage	pH	NTU Turbidity Treated Water
0	7.3	18
0.5	7.3	7.0
1	7.3	6.0
2	7.3	1.8
3	7.3	0.55
5	7.3	0.44
8	7.3	0.39

The polymer action appeared to be similar to that of alum. A dosage equal to 3 mg/l was enough to ensure excellent results.

TREATMENT TRIALS

After completing a series of preliminary tests, several testing cycles were carried out so as to verify the practical aspects of the treatments such as cycle length, quality of water, pressure drops, chlorination effects and so forth. To start backwashing the general parameters considered in order to establish the end of a cycle were both turbidity leakage $\gg$ 1 NTU and flow decrease beyond 10% - 15% of the initial flow due to pressure drops. The filters therefore operated on a declining rate. Six different procedures were tested out in relation to chemical dosages and filtration velocities. These procedures have been summarized in Table 5.

Table 5 - Tests Carried Out

Test No.	Flow Rate m^3/h	Filtration velocity $m^3/m^2 \cdot h$	Chlorine Dosage	Chemical ahead I filter	Chemical ahead II filter
1	4.0	20	YES	Alum 20 mg/l	Non-ionic 0.02 mg/l
2	4.0	20	YES	Alum 10-15 mg/l	Non-ionic 0.02 mg/l
3	4.0	20	NO	Cationic 3 mg/l	Non-ionic 0.02 mg/l
4	4.0	20	NO	Alum 15-10 mg/l	Alum 10 mg/l
5	3.0	15	YES	Alum 10 mg/l	Alum 10 mg/l
6	3.0	15	NO	Cationic 1.5 mg/l	Cationic 1.5 mg/l

With the exception of trial No. 2, the qualitative results were excellent. The outcome of trial No. 2 was, however, up to the expectations as it was run with little alum injected. In comparison with trial No. 1, it highlighted the specific action of non-ion polymer exerted on turbidity caused by a breakthrough of destabilized particles whereas its action has no effect in case of particles that are still probably colloidal. The pressure drop being the most critical parameter, the most considerable difference in velocity is that between 20 $m^3/m^2 \cdot h$ and 15 $m^3/m^2 \cdot h$.
The following diagram (4) shows the trend of turbidity leakages from the first to the second filter during the 1st, 3rd and 4th trial as they shared a high filtration velocity in common.

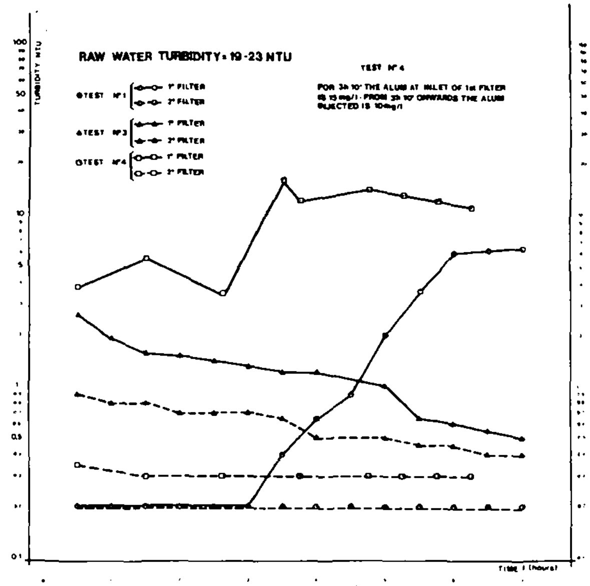


Diagram 4 - Trend of turbidity leakages from the 1st to the 2nd filter during the 1st, 3rd and 4th trial. Filtration velocity 20 $m^3/m^2 \cdot h$

The pressure drops and ensuing decreases in flow rate relevant to above mentioned trials are shown in diagram 5.

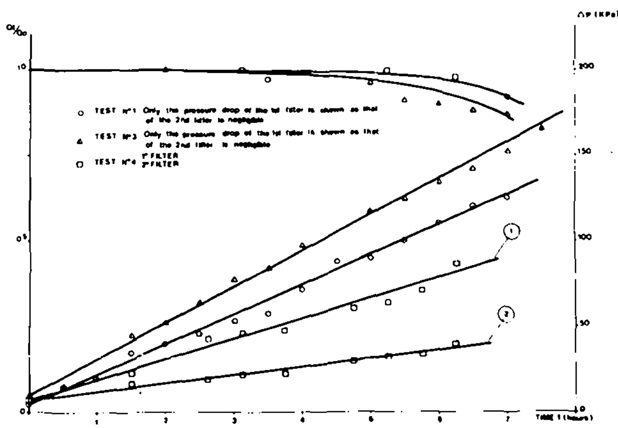


Diagram 5 - Pressure drop and flow decrease as a function of time during the 1st, 3rd and 4th trial.

This series of trial gave rise to the following observations:

- Clarification and dechlorination are proportionally incomplete when the dosage of coagulant goes below the given threshold value.
  Dosages in excess do not improve the situation. Presumably it is worsened owing to pressure drops.
- The injection of Alum at the inlet of the 1st filter causes a turbidity leakage from a given point onwards. The turbidity leakage is however buffered by the 2nd filter. This is the standard operational approach to turbid water by Ofsy.

- Cation polymer acts similarly to alum when removing colour and turbidity. However the trend of turbidity leakage and the increase in pressure drop demonstrate the clogging property of the strong flocs of cation polymer.
- Relatively high pressure drops did not entail sharp decreases in flow rate. Flow rate decreases are negligible at the beginning of the cycle and only start to become important towards the end of it. It may occur that the laminar flow of the water inside the filter becomes turbulent. This hypothesis is not corroborated by a steady and linear increase in pressure drops.
- The splitting of alum injection between the first and second filter favourably contributed to limiting a decrease in flow rate, other conditions being equal.

Diagram 6 shows the trend of turbidity leakage relevant to trial No. 5 and No. 6 at a filtration velocity of 15 $m^3/m^2 \cdot h$

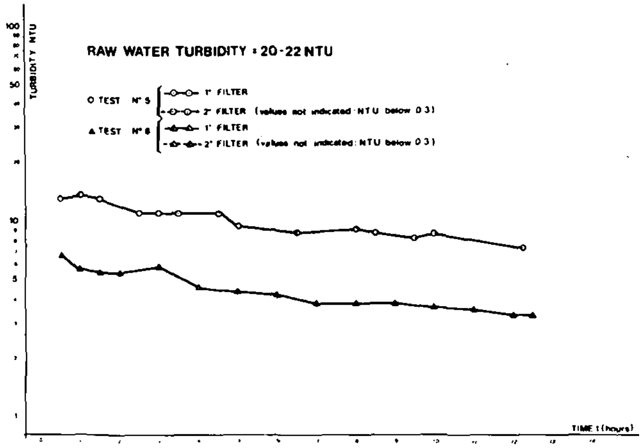


Diagram 6 - Trend of effluent turbidity from 1st and 2nd filter during tests Nos. 5 & 6

Diagram 7 shows the pressure drops and flow rate decreases relevant to trial Nos. 5 & 6.

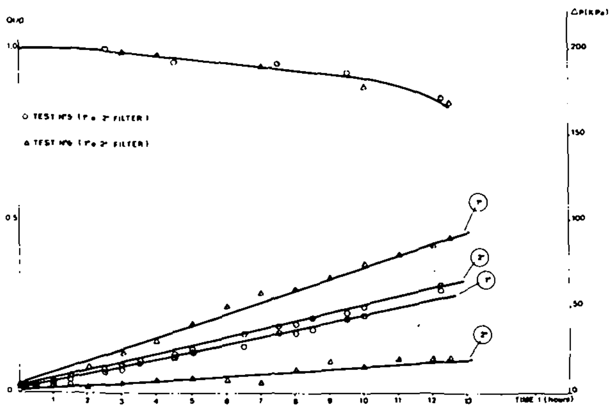


Diagram 7 - Pressure drops and flow rate decrease (trials Nos. 5 & 6)

The previous considerations apply to these latter tests as well. Further considerations to be pointed out are the following:

- The total pressure drop being the same and the flow rate decrease approximately corresponding to it, the drop in filtration velocity and above all the subdivision of pressure drops between the two filtering units proved to be advantageous specially when cation polymer was injected instead of alum.
- The length of the cycle mainly depends on the maximum permissible pressure drop.

OTHER CHEMICAL PARAMETERS

Besides turbidity and pressure drop, other parameters have been kept under control so as to attest the efficacy of the treatment. Table 6 summarizes the results obtained by making an average of the values relevant to several samples drawn at the outlet of the system during each cycle.

Table 6 - Chemical and Chemico-Physical Results of the Treatment.

Parameter	Raw Water	Run No. 1	Run No. 3	Run No. 4	Run No. 5	Run No. 6
pH	7.35	7.0	7.35	7.0	6.95	7.35
Turbidity	20	0.2	0.5	0.25	0.23	0.25
Apparent Colour	45	1	2	1	1	1
True Colour	16	-	-	-	-	-
Conductivity	88	117	97	105	105	100
Iron	1.45	0.02	0.01	0.01	0.01	$<$ 0.01
Manganese	0.06	$<$ 0.01	$<$ 0.01	$<$ 0.01	$<$ 0.01	$<$ 0.01
Aluminium	0.14	$<$ 0.01	$<$ 0.01	$<$ 0.01	$<$ 0.01	$<$ 0.01

CONCLUSIONS

The tests provided the demonstration that colour can be removed by direct filtration with an arrangement of in-series filters. Moreover the following conclusions can be drawn:

- In order to remove colour it is necessary to achieve a stoichiometric relation with the alum injection. The quantity of alum to be injected is however halved as compared with that usually employed by conventional treatments.
- In-series filtration by Ofsy makes it possible to obtain results that are not easily attained by other systems based on direct filtration. As a matter of fact it can buffer leakages of varying turbidity and makes possible the subdivision of the polltuing load by means of a careful distribution of the coagulants.
- Contrary to expectations and other experiments in colour removal from surface water (i.e. Norway), the pH has little affect on the outcome.
- During the test the pH conditions limited aluminium leakage to negligible values even iron and manganese were almost completely eliminated.
- By eliminating the precursors from water, the possibility of chlorinating treated water without giving rise to the undesired formation of THM or halogen derivatives may be envisaged.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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EVOLUTION RECENTE DE LA LEGISLATION SUR LES EAUX SOUTERRAINES EN BELGIQUE

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RESUME

En Belgique, la législation actuelle concernant la gestion des eaux souterraines est basée sur quatre lois qui traitent respectivement de l'aspect quantitatif (loi du 18 décembre 1946 et 9 juillet 1976), de l'aspect qualitatif (loi du 26 mars 1971) et de l'aspect "influence-dommage" (loi du 10 janvier 1977).

Ces lois sont opérationnelles sauf la loi du 26 mars 1971 qui, à défaut d'arrêtés d'exécution, n'est pas efficace. En plus leur gestion ressort de la compétence de deux Ministres.

La loi spéciale de reformes institutionnelles du 8 août 1980 institue une gestion des eaux relevant de la compétence régionale. Depuis lors les différents aspects de la gestion des eaux souterraines dans la Région nord sont regroupés dans un seul décret du 24 janvier 1984 et placés sous la compétence d'un seul Ministre.

Mots-clés : Eaux souterraines - législation - dommage - prise d'eau - zone de protection.

INTRODUCTION.

L'homme a une meilleure connaissance des eaux de surface par rapport aux eaux souterraines ne fût-ce que par le fait de l'observation directe des eaux de surface. C'est peut-être une des raisons pour laquelle la législation sur les eaux de surface est plus développée que celle des eaux souterraines. H.E. Thomas confirmait cette constatation en disant que l'hydrologie serait une science relativement simple si l'eau ne disparaîssait pas dans le sous-sol. La même constatation équivaut au niveau législatif.

LEGISLATION ACTUELLE.

En Belgique la législation actuelle concernant la gestion des eaux souterraines est basée sur quatre lois qui traitent respectivement de l'aspect quantitatif, de l'aspect qualitatif et de l'aspect "influence-dommage".

- La loi du 18 décembre 1946 (publiée au Moniteur belge le 6 mars 1947) vise le recensement des réserves aquifères et une réglementation de leur usage. Cette loi est devenue opérationnelle suite à la promulgation d'une série d'arrêtés d'exécution e.a. les arrêtés royaux du 21 juin 1976 (modifié par l'A.R. du 5 juin 1978), du 14 juin 1966 et de l'arrêté ministériel du 21 novembre 1973, traitant respectivement des autorisations de prélèvement d'eau, du recensement des ressources aquifères souterraines et des dispositifs de comptage de prises d'eau.

- La loi du 9 juillet 1976 (publiée au Moniteur belge du 28 août 1976) prévoit la possibilité d'imposer de nouvelles conditions d'exploitation aux prises d'eau souterraine antérieures au 15 juillet 1947 et dont le débit journalier est supérieur à 96 m³. Cette loi est devenue opérationnelle suite à la promulgation d'une série d'arrêtés d'exécution e.a. les arrêtés royaux du 1 octobre 1976 et du 9 août 1976 respectivement relatifs à la règlementation de leur usage et à leur recensement.

- La loi du 26 mars 1971 sur la protection des eaux souterraines (publiée au Moniteur belge du 1 mai 1971) vise surtout la protection de la qualité de l'eau souterraine dans le cadre de la santé publique. Les directives principales concernaient la possibilité de délimiter des zones de captage et des zones de protection, d'interdire ou de réglementer des activités dans ces zones et aussi, si nécessaire, en dehors de ces zones. Aussi, l'article six stipule que : "Les dommages directs et matériels subis par le propriétaire ou l'exploitant et résultant d'une prise en exécution de l'article 2.2, sont réparés au frais du bénéficiaire de la protection".

  A titre d'information il faut mentionner la loi du 12 juillet 1973 (publiée au Moniteur belge du 11 septembre 1973) tendant à sauvegarder le caractère, la diversité et l'intégrité de l'environnement naturel e.a. les nappes aquifères.

- La loi du 10 janvier 1977 organise la réparation des dommages provoqués par des prises et des pompages d'eau souterraine (publiée dans le Moniteur belge du 8 février 1977).

  Abstraction faite de la surexploitation des nappes aquifères elles-mêmes, l'abaissement du niveau des nappes peut provoquer des dommages aux immeubles de surface, y compris le sol et la végétation. Les principes sont que l'exploitant de la prise d'eau souterraine est responsable du dommage qui résulte de l'abaissement du niveau de l'eau souterraine, qu'une procédure spéciale est prévue pour le dédommagement et qu'on a créé un "Fonds d'avances pour la réparation des dommages provoqués par les prises et pompages d'eau souterraines". Ce fonds est alimenté par des contributions payées par les exploitants d'eau souterraine. Cette loi est opérationnelle depuis le 6 décembre 1978 et le Ministre des Affaires Economiques en est responsable de la gestion.

En bref la situation actuelle est la suivante :

- les lois du 18 décembre 1946, du 9 juillet 1976 et du 10 janvier 1977 sont opérationnelles contrairement à celle du 26 mars 1976. Hanotiau (1977) a commentarié cette dernière disant que "la loi est entrée en vigueur le premier mai 1971, mais son efficacité en tant que loi-cadre et à défaut d'arrêtés d'exécution, est extrêmement réduite, pour ne pas dire nulle". Depuis 1977 la situation n'a pas évolué.

- la protection quantitative (les lois du 18 décembre 1946, du 9 juillet 1976 et du 10 janvier 1977) ressort de la compétence du Ministre des Affaires Economiques. La protection qualitative (la loi du 26 mars 1971) ressort de la compétence du Ministre de la Santé Publique.

Par conséquent deux administrations s'occupent de l'exploitation de l'eau souterraine en utilisant une législation à portée réduite au moins quant à l'aspect qualitatif.

MODIFICATION DE LA LEGISLATION ACTUELLE.

La loi spéciale de réformes institutionnelles du 8 août 1980, instituant que la gestion des eaux est confiée à la compétence régionale, a modifié la situation.

A l'occasion de cette loi, les quatre lois précitées ont été reprises par le décret du 24 janvier 1984, portant des mesures en matière de gestion des eaux souterraines. Ce décret a été élaboré pour la Région nord du pays et n'a pas encore été publié au Moniteur. Pour la Région sud la législation nationale est toujours en vigueur.

La gestion des eaux souterraines ressortira uniquement de la compétence du Ministre communautaire (régional) de l'Environnement, de la Politique de l'eau et de l'Enseignement. Dès que ce décret entrera en vigueur, la législation actuelle sera abrogée dans la partie nord du pays. Ce décret comporte six chapitres traîtant respectivement de :

- la protection des eaux souterraines.
  Les principes de la loi du 26 mars 1971 ont été conservé c.à.d. les délimitations des zones de protection, les mesures à prendre à l'extérieur et à l'intérieur de ces zones, la responsabilité de l'exploitant et le droit de dédommagement lors de mesures à l'intérieur des zones de protection.

- la réglementation de l'usage des eaux souterraines.
  Les principes de la loi du 18 décembre 1946 et du 9 juillet 1976 sont conservés c.à.d. la réglementation de l'usage et du recensement des ressources en eau souterraine.

- la surveillance sur l'observation et l'application de la législation.

- la prévention et l'indemnisation des dommages. Les principes de la loi du 10 janvier 1977 sont conservés (responsabilité, dédommagement, fonds).

- les dispositions pénales.

- les modalités de transistion.

Outre une fusion des lois existantes et des administrations responsables, ce décret apporte quelques modifications essentielles :

- L'article 3, § 1.1° du décret du 24 janvier 1984 dit que "Afin de protéger les eaux souterraines, en vue de leur utilisation éventuelle à des fins alimentaires, l'Exécutif flamand peut prendre les mesures suivantes : ... interdire, réglementer ou soumettre à autorisation, dans toute la Région flamande, le déversement direct ou indirect, le dépôt, le stockage sur ou dans le sol, de matières susceptibles de polluer les eaux souterraines.".
  Au lieu de concevoir la protection des prises d'eau souterraine à partir des prises elles-mêmes, la législation nouvelle part de la conception que toute la surface d'une Région doit être protégée conformément à la directive du 17 décembre 1979 de la Communauté Européenne concernant la protection des eaux souterraines contre la pollution causée par certaines substances dangéreuses (80/68/CEE). Plus on se rapprochera d'une prise d'eau, plus les règlements entrant en vigueur seront stricts. En plus les sociétés de distribution d'eau délimiteront elles-mêmes leur zones de protection et seront responsables des suites qui en découlent.

- L'article 4 du décret stipule qu 'il est défendu de déverser ..., de déposer ou de stocker sur ou dans le sol, des matières fécales ... sous forme liquide provenant d'un territoire extérieur à la Belgique" et que des dérogations peuvent être accordées. Un des arrêtés mentionnés ci-dessous porte sur cette dérogation.

- L'article 6 de la loi du 21 mars 1971 traite du droit au dédommagement pour dégât matériel comme suite directe des mesures de protection prises éventuellement.

  D'après l'article 8 de ce décret, ce dédommagement est maintenant bien limité aux installations ou activités existantes qui, suite à ces mesures de protection, doivent être arrêtées, réduites ou reconverties et cela pour autant que le bénéficiaire de la protection en fasse la demande.

- L'article 14 du décret stipule que l'exploitant d'une prise d'eau souterraine et le maître de l'ouvrage de travaux publics ou privés sont objectivement responsables des dommages de surface qui sont causés aux prises d'eau souterraine existantes et à d'autres immeubles, y compris le sol et la végétation.

  D'après le § 2, ces dispositions concernant la responsabilité des dommages ne sont pas applicables aux dommages résultant de travaux d'exhaure dans les mines. Il existe d'ailleurs une autre réglementation satisfaisante quant au dédommagement des dégâts miniers.

  La différence entre la situation future et la situation existante réside dans le fait que les carrières et minières deviendront responsables pour les dégâts résultants de l'exhaure de l'eau souterraine.

Quelques huit arrêtés relatifs aux articles du décret du 24 janvier 1984 seront publiés sous peu :

- Arrêté portant sur la réglementation et l'autorisation des acitivités susceptibles de polluer l'eau souterraine dans toute la région flamande ou dans certaines parties de celle-ci.

- Arrêté portant la délimitation des zones de protection et de captage des eaux souterraines.

- Arrêté portant la réglementation des activités dans les zones de captage et de protection.

- Arrêté fixant les conditions auxquelles sont accordées les dérogations visées à l'article 4 du décret du 24 janvier 1984.

- Arrêté portant sur la réglementation et l'autorisation de l'usage de l'eau souterraine.

- Arrêté désignant les agents compétents pour rechercher et constater les infractions portant des mesures en matière de gestion des eaux souterraines.

- Arrêté fixant les modalités de l'échantillonnage dans le cadre de la recherche et de la constatation des infractions.

- Arrêté fixant les conditions d'agréation des laboratoires.

CONCLUSION.

Les nappes aquifères constituent un patrimoine à défendre aussi bien sur le plan quantitatif que qualitatif. La promulgation du décret du 24 janvier 1984, synthétisant le contenu des quatre lois existantes et plaçant la gestion des eaux souterraines pour la Région nord sous la tutelle d'un seul Ministre, donc d'une seule administration, en simplifiera la gestion et la rendra plus efficace.

L'obtention d'un outil de travail législatif dans le cadre d'une gestion intégrale des eaux souterraines constitue une évolution très importante vers une politique de protection efficace de l'environnement. Ainsi l'eau sera mieux protégée contre les risques de pollution et les menaces d'épuisement et ce particulièrement pendant son séjour souterrain où elle est vulnérable de manière irréversible.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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ECONOMIC TOOLS TO AID RURAL WATER DECISIONMAKERS

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ABSTRACT

A major determinant of the quality of life in rural areas is an abundant supply of high quality water for domestic use. Agricultural and industrial water requirements must also be met if rural areas are to flourish. Rural residents have for years relied on groundwater or have hauled water for their needs. Some rural areas do not have adequate supplies of quality water. Indications are that the water supply problem will continue or perhaps become worse in the future due to population growth. Community leaders and rural water district boards are particularly concerned with the water issue. Several problems confront these leaders as they attempt to plan and develop water supply and distribution systems to adequately meet their present and future needs. Several research projects have been undertaken to develop economic tools which will aid these leaders. They include: (1) a method to estimate future water use based on historical water use trends, socio demographic data and population projections; (2) a method to estimate annual capital and operating budgets; and (3) a tool to use to evaluate alternative water rate structures and their impact on revenue.

Keywords: Planning, rural water systems, water demand, budgets, rate schedules, demand elasticity.

A major determinant of the quality of life in rural areas is an abundant supply of high quality water for domestic use. A front page story in The Daily Oklahoman (July 2, 1984) clearly illustrates this fact: Mayor Thomas Blankenship from Sasakwa, Oklahoma, a rural community of 405 residents, reports that two water wells were recently drilled and the water did not meet State Health requirements. The solution appears to be to drill a well five miles away and pipe the water to the city. The costs of this alternative is much more than the community can afford. The Mayor concludes "This little town would grow if we could supply water, but I'm afraid it's too late."

Community leaders and rural water district boards are particularly concerned with the water issue. These leaders seek information as to how to plan and develop water supply and distribution systems to adequately meet their present and future needs. Several research project have been completed which have developed economic tools which aid these leaders. The objective of this paper is to summarize these economic tools. These include: 1. a method to project present and future water use; 2. a tool to estimate annual capital and operating budgets; and, 3. a method to evaluate alternative water rate structures. The tools will be briefly discussed and illustrated.

PROJECTING WATER USE

Many capital investment decisions, are based on current and future demand for water. Thus, estimates of future water demand are imperative. Furthermore, engineering data can be used to estimate the life of various water sources but water demand data are needed to select the source which will provide a sufficient quality and quantity of water. Demand for water is defined as the various quantity of water a consumer is willing and able to buy at a given price ceteris paribus. In rural water districts, a household is considered the consuming unit. Water system demand is the summation of the demand of all consumers in the district. Demand elasticity is the relationship between price and quantity demanded.

Data and Model

Most studies analyze water needs based on rural water district data. The cross-sectional data utilized in this analysis were obtained from responses to a 1982 mail survey sent to 2,159 RWD customers in seven counties in east central Oklahoma (2). Of those 1,139 responses received, 658 contained complete information and were analyzed in this particular study. The mean monthly water use was 5,704.33 gallons and the mean marginal price was 0.2625 per 100 gallons.

Ordinary least squares (OLS) regression was selected as an appropriate method to analyze the data. For purposes of this research, the monthly water use per customer was functionally specified as follows: Monthly water demand = f (number of persons in household, presence of modern conveniences, price of water, educational attainment of household head, non-domestic water uses, annual family income). Hypotheses concerning each variable's relationship to monthly water use are:

1. Number of persons in household (NOPERS): An increase in the number

of persons per residence is hypothesized to increase the amount of water used.

2. Modern conveniences (BUILT): Previous research such as by Burns and Goode (1980) indicates that use of such modern conveniences as washing machines, dishwashers and garbage disposals, as well as additional bathroom and shower facilities, contribute to a larger water use. Inclusion of these conveniences in homes built in more recent years is more common. Therefore, the year in which the residence was built is used as a proxy for their presence.

3. Price (MC): The marginal cost (MC) of water to the customer (dollars per 100 gallons) is used to represent price in the analysis. MC was calculated for each customer by taking the number of gallons used per month and applying that to the appropriate rate structures for each customer's district.

4. Education (TOTED): In general, people with a higher level of education tend to demand better services and more conveniences and therefore are hypothesized to use more water per month than other customers.

5. Water for stock (STOCK1, STOCK2): Cattle and horses are often watered from water provided by the RWD. STOCK1 and STOCK2 are hypothesized to have a positive relationship.

6. Annual family income (Y): Household income is hypothesized to be positively related to the total gallons of water used in the home per month. Data for incomes were collected for seven income ranges so that Y received a value equivalent to the midpoint of the income ranges.

Empirical Results

Several models were tested in analyzing the data obtained from survey responses. In all of these, all variables except Y and MC (price of water) were shown to be linear. Several functional forms of the income and price variable were tried. Results for models employing inverse and log-linear forms of these variables are reported here. These are the most appropriate in estimating monthly per customer water demand on the basis of statistical reliability and economic consistency. Results are presented in Table 1.

Coefficients on all variables except BUILT are statistically significant at a level of at least 0.05, and the overall R^2 values are 0.296, 0.291, 0.283 and 0.272 for equations 1 - 4, respectively. Specifically for equation 1, the coefficient on the income variable (Y) states that for each dollar increase in a customer's annual income, monthly water consumption would increase by 0.03 gallons. The coefficient for the number of persons in the household variable (NOPERS) states that each additional person adds 871 gallons to monthly water consumption, *ceteris paribus*. The coefficient of the variable which is a proxy for modern conveniences (BUILT) is not significant. The proxy, which was the year the home was constructed, may not adequately measure the presence of modern conveniences. The coefficient for the education variable (TOTED) states

TABLE 1. Summary of Water Demand Estimation Equations for Rural Water Service Areas

Variables	Equation 1 Coefficients	Equation 2 Coefficients	Equation 3 Coefficients	Equation 4 Coefficients
Y^b	0.030067	443.552	0.03378	489.756
	(2.64)[a]	(1.78)	(2.93)**	(1.96)*
NOPERS	870.954	854.714	899.327	885.142
	(6.67)**	(6.52)**	(6.85)**	(6.68)**
BUILT	7.876	7.431	10.372	9.608
	(0.75)	(0.69)	(0.91)	(0.88)
TOTED	177.578	190.95	183.179	196.455
	(2.78)**	(2.93)**	(2.89)**	(2.98)**
STOCK1	79.595	81.765	82.668	84.031
	(3.28)**	(3.36)**	(3.39)**	(3.41)**
STOCK2	93.392	105.6	103.681	114.419
	(1.51)	(1.71)	(1.68)	(1.83)
MC^c	609.471	628.963	-2247.446	-2309.438
	(7.10)**	(7.34)**	(-5.91)**	(-6.19)**
INTERCEPT	-3820.09	-7613.33	-4442.82	-8673.416
	(-4.14)	(-3.59)**	(-4.61)**	(-4.05)**
R^2	0.296	0.291	0.283	0.272

[a]Numbers in parentheses represent the observed "student-t" values. "*" denotes an observed significance level (OSL) greater than or equal to 0.05. "**" denotes an OSL greater than or equal to 0.01.
[b]Equations 1 & 2 specify MC as an inverse function; equations 3 & 4 specify MC as log linear.
[c]Equations 1 & 3 specify Y as linear; equations 2 & 4 specify Y as log linear.

that as education for the household head increases by one year, monthly water consumption increases by 177.58 gallons. Coefficients on STOCK1 and STOCK2 indicate that each cow adds 79.6 gallons per month and each horse adds 93.4 gallons per month to water consumption, respectively. The inverse of price specifies that for each unit increase in the inverse of marginal costs (1/MC), monthly water consumption will increase 609.47 gallons.

To isolate the effect of price and income on water demand, price elasticities of demand and income elasticities were calculated for the four model specifications previously presented. General formulations for price and income elasticities may be shown as follows:

Price Elasticity of Demand: $\varepsilon_D = \partial Q/\partial P \cdot \overline{P}/\overline{Q}$ $\quad \overline{P} = 0.2625;\ \overline{Q} = 5704.33$

Income Elasticity: $\varepsilon_Y = \partial Q/\partial Y \cdot \overline{Y}/\overline{Q}$ $\quad \overline{Y} = 25{,}760;\ \overline{Q} = 5704.33$

where: Q = quantity of water demand per month; P = price (marginal cost per 100 gallons) of water; and Y = annual family income.

Price and income elasticities calculated at the variable means over the relevant range of data for each model specification are presented in Table 2, along with estimates of monthly per customer water demand. These calculations indicate that a price change does have an impact on the quantity of water demanded in rural areas. In all cases, regardless of functional form, the price elasticity of water is somewhat inelastic. Elasticities of demand near -0.4 imply that for a one percent increase in the price of water, the consumption of water will decrease by 0.4 percent, all other things remaining unchanged.

Income elasticities for the four models vary from 0.078 to 0.153. These values indicate that the monthly quantity of water demanded per customer is relatively unaffected by annual family income over the range of the sample. For example, an income elasticity of 0.153 indicates that for a one percent increase in income, water consumption increases by 0.153 indicates that for a one percent increase in income, water consumption by 0.153 percent. This income inelastic behavior is to be anticipated due to the nature of water as a necessary good.

Additionally, it is seen from Table 2 that monthly water consumption is estimated to be from 4,900-5,430 gallons per customer. These estimates were derived from utilizing county mean data for each variable and substituting to arrive at the final quantity.

The empirical results are useful to rural water boards as they estimate future water needs and evaluate water pricing policies. If a board is considering a short-term question such as expanding the system to include

TABLE 2. Estimated Quantity Demanded and Demand and Income Elasticities for Water in Rural Service Areas[a]

Parameter	Equation 1	Equation 2	Equation 3	Equation 4
Q_D	4900.11	5015.15	5408.21	5430.61
ε_D	-0.407	-0.420	-0.394	-0.405
ε_Y	0.136	0.078	0.153	0.086

[a]Quantity demanded is expressed in gallons per month per customer calculated at county mean values for all variables except income. Sample mean is used for income.

new customers, a simple estimate of water needs can be obtained from the mean consumption per month, which is 5,704 gallons per hookup. If the board asked several simple questions when the customer applies for service, the results of regression analysis can be sued to estimate water usage for each new customer. The water consumption estimate as well as future population must be taken into account when considering the issues of size of water line, and sizes of water source, storage and treatment plants large enough to serve existing and future customers.

The study results, although extremely useful for short-term analysis, are also needed for long-term planning. Water usage estimates, trends in water usage and population estimates are all important variables for estimating long-range water needs such that large capital investments (e.g., a treatment plant) will be constructed appropriately to serve the rural water district.

ESTIMATING ANNUAL CAPITAL AND OPERATING COSTS

After current and future water demand is projected, local decisionmakers can determine the desired size of the water supply, treatment and distribution. With this information, annual capital and operating budgets can be derived. The method used to estimate capital and operating costs were developed by Goodwin et al. (1979) and is updated with the CPI and CCI indexes. Information and procedures were developed to estimate community or area water needs by types of users and, capital and operating budgets for alternative systems. Data from 82 water systems were obtained from the Oklahoma State Office of the Farmers Home Administration (FmHA) for use in the study. The results of this study have been utilized numerous times in Oklahoma to evaluate, ex ante, the economic feasibilities of planned water systems or system expansions. "Easy-to-use" forms were developed in the study. These forms can be utilized along with local data and data reported in the study to estimate costs and revenues for community and area water systems.

Decisionmakers for a small town in Oklahoma were considering the construction of a local water system. They requested information from Oklahoma State University personnel concerning the amount of revenue the community would find necessary to generate in order to finance and operate such a system. Cost estimates shown in the completed set of forms presented were included in a brief report which was presented to local decisionmakers by Nelson et al. (1981). Using FORM 1, total capital costs for a water system to serve the example town were estimated as almost $430,350. Total annual operating costs for the system were estimated as about $36,108 (FORM 2). Annual capital costs were, estimated on FORM 3 for alternative grant-loan combinations (based on then existing FmHA programs), assuming a 5%, 40 year FmHA loan. These costs were estimated to range from $12,535 per year to $25,319 per year. Any grant-loan combinations and interest rate-payback period options thought to be appropriate could have been considered.

Total water system costs and total costs per user, for each of the alternative financing options considered were estimated in FORM 4. Monthly costs per user were estimated as ranging from $10.66 to $13.92, depending on the grant-loan option considered. Estimated total monthly costs per user can be evaluated relative to a water rate which local decisionmakers feel is acceptable.

Methods for analyzing rural water systems tend to be rather complex. While these methods may not be recognized as overly sophisticated by professional economists, they can be fraught with difficulty and confusion when applied by community leaders, public agency personnel or extension

FORM 1
Sample Capital Costs Budget for Rural Water Systems Construction

Type of System WELL

Residential Users 323
Commercial Users ____

A. Lines (Materials, Placement & Labor):

Pipe Description Type and Strength	Size	No. Feet		Cost/Unit		Total Costs
1. 200 PVC	6"	5800	X	4.69	=	$27,202
2. 200 PVC	4"	6300	X	2.54	=	16,002
3. 160 PVC	3"	4200	X	1.54	=	6,468
4.			X		=	
		SUBTOTAL A =				$49,672

B. Crossings (Materials, Placement & Labor):

	Number		Cost/Unit		Total Costs
1. Highway (Paved)		X		=	
2. Country Road (Unpaved)	21	X	344	=	$ 7,224
3. Railroad		X		=	
4. Stream	6	X	3465	=	20,790
5. Turnpike		X		=	
	SUBTOTAL B =				$28,014

Sections C (Storage Facilities), D (Valves and Meters), E (Pumping and Well Facilities) and F (Other Construction Costs) were not included in paper due limited space, see Goodwin et al. (1979) for details.

TOTAL CONSTRUCTION COSTS (A + B + C + D + E + F) = ____

G. Legal (3% Construction Cost)	G =	$ 9,127
H. Engineering (7% Construction Cost)	H =	21,296
I. Interest During Const.(2 1/2% Const.Cost)	I =	7,606
J. Inspection (2% Construction Cost)	J =	6,085
K. Contingencies (5% Const. Cost)	K =	15,326
TOTAL PROJECT COSTS (Add items A through K)	=	$361,373

$$\text{Total Project Costs} \times \frac{\text{Curr.Const.Cost Index}}{\text{1977 Const.Cost Index}} = \text{Total Proj.Costs in Current \$\$}$$

$361,373 X 249.1/209.2 = $430,035

****

FORM 2
Sample Operating Costs Budgets

Type of System WELL

Residential Users 323
Commercial Users ____

	Cost/User		Total No. Users		
Wages		X		=	$ 9,709
Utilities		X		=	3,918
Office		X		=	940
Insurance & Bonds		X		=	1,447
Taxes		X		=	623
Professional Fees		X		=	1,544

Repairs		X		=	4,438
Water Purchases		X		=	1,680
Miscellaneous		X		=	4,477
			TOTAL		#28,776

Total Operating Costs X Curr.Consumer Price Index / 1978 Consumer Price Index = Total Operating Costs in Current $$

#28,776 X 307.3/244.9 = $36,108

****

FORM 3

Repayment Schedule for FmHA Loan for Expansion or Establishment of a Rural Water System

Type of System WELL

Residential Users 323

Commercial Users ______

	Case I	Case II	Case III
	0% Grant 100% Loan	25% Grant 75% Loan	50% Grant 50% Loan
A. Total Project Cost (Current Dollars)	#430,035	#430,035	#430,035
B. Initial Service Charge			
1. Residential - $____/meter X ____users			
2. Commercial - $____/meter X ____users			
C. Total Funds Needed (A-B1-B2)	#430,035	#430,035	#430,035
D. Grant Funds Received	0	107,509	215,018
E. Amount of Loan Required (C-D)	#430,035	#322,526	#215,017
F. Ammortization Factor (for appropriate interest rate and payback period for loan)	0.0583	0.0583	0.0583
G. ANNUAL PAYMENT (E x F)	#25,319	#18,803	#12,535

****

FORM 4

Total Water System Costs

Type of System WELL

Residential Users 325

Commercial Users ______

	Case I	Case II	Case III
	0% Grant 100% Loan	25% Grant 75% Loan	50% Grant 50% Loan
A. Annual Capital Costs	#25,319	#18,803	#12,535
B. Annual Operating Costs	28,776	28,776	28,776
C. Total Annual Costs	#54,095	#47,579	#41,311
D. Annual Costs per User	167	147	128
E. Monthly Costs per User	#13.92	#12.25	#10.66

****

field personnel from universities where training is oriented more toward interaction with people than toward analytical techniques. The application to local circumstances of analytical techniques developed and validated in research situations can be very time consuming. Local community leaders, agency personnel or field extension specialists with numerous other responsibilities may be hard pressed to localize such models, operationalize them, and produce results when working with a desk calculator and a pencil under severe time constraints.

To reduce time demands and simple mathematical errors, a computer program written by Nelson and Mostafavi (1981) such that the analysis completed in the previous section can be completed in minutes. The computer program is interactive and written for IBM mainframe or Radio Shack computers. Thus, a portable terminal can be used any place in the state or nation, and hooked into the mainframe to complete the analysis on location. Likewise, a Radio Shack computer can be taken to the community to analyze the problem on site in minutes. By having the program on site with local decisionmakers, alternatives can be analyzed quickly. The interactive computer program is very easy to operate. It requires no computer experience and anyone can complete the analysis by answering the questions.

A TOOL TO EVALUATE ALTERNATIVE WATER RATE SCHEDULES

After estimating annual capital and operating costs of the desired system, local decisionmakers need to select a rate schedule. Local decisionmakers desire to select a rate schedule which is equitable, but which also generates sufficient capital to cover annual capital and operating costs. A computer program has been written which allows local decisionmakers a rapid evaluation of alternative rate structures. To make the program operable, data must be entered on number of users and consumption by user for each month in a year. Alternative rate structures can be specified and the computer will calculate total revenue which would be generated from users.

To illustrate usage of the program, a select portion of study for Ottawa County Rural Water District 2 will be presented. The district board members wanted to compare revenues from alternative rate structures. A computer terminal was taken to their monthly meeting and rate schedules were analyzed there. A sample of the output is presented in Figure 1. After seeing one rate schedule, the board members would say, what if we changed this portion of the schedule. The board continued to change rate schedules until they found a schedule which they felt was equitable and which generated sufficient revenue to cover costs.

SUMMARY

Many of the problems faced by rural water boards or city councils as they plan rural water systems are economic problems. Economists have an assortment of tools which can be used to address these problems. Three tools were summarized in this paper and include: a method to project water demand; a procedure to estimate annual capital and operating expenses; and a method to evaluate alternative water rate schedules. The usefulness of these tools is demonstrated by the fact that during the past several years, the authors have used the tools to assist leaders in many rural Oklahoma communities and rural water districts.

Total Water Sales (Dollars) by Month and Use Class

Less Month	1500 Than 1500	4500 To 4500	6500 To 6500	More To 10000	Than 10000	Total
J	803	1281	398	293	125	2899
F	548	565	457	799	1979	4347
M	638	883	677	489	726	3412
A	585	750	604	866	1144	3948
M	525	784	716	753	1051	3831
J	510	761	559	929	815	3574
J	503	620	711	965	878	3677
A	548	601	562	1054	1055	3820
S	578	750	666	826	838	3657
O	623	534	661	943	971	3731
N	683	757	639	777	609	3464
D	660	926	675	660	446	3366
AVG	600	768	610	779	887	3644

TOTAL ANNUAL DOLLARS: $43,726.89

FIGURE 1. Information Above is Based on the Following Price Scheme:

Under 1500 gallons: flat rate of $7.50
Next 3000 gallons are charged at: $.110 per 100 gallons
Next 2000 gallons are charaged at: $.080 per 100 gallons
Next 3500 gallons are charged at: $.075 per 100 gallons
Above 10,000 gallons charged at: $.070 per 100 gallons

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MITIGATION OF DROUGHT IMPACTS

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ABSTRACT

The intention of this paper is an analysis of the different measures and strategies for the mitigation of drought impacts, with particular consideration of the different ways in which normal water strategies should be modified when drought conditions begin to develop or are already well established. Some basic drought concepts are presented, including definitions and references to the concepts of aridity and drought forecasting. A detailed analysis of the several possible drought impacts is made, presenting an enumeration of possible economic, social and environmental impacts and giving some comments on specific aspects of these impacts. The possible responses to drought situations are identified and the measures for drought impact control are classified in three groups: measures intended to increase the available water supply during the drought periods, measures intended to decrease the water demand during the drought periods and measures intended to minimize drought impacts which still occur after the application of the first two types of measures. Final comments on the implementation of these three types of measures are given.

INTRODUCTION

Drought impacts have been felt with growing intensity in several regions. Therefore, their identification, the study of their impacts and the mitigation of these impacts have been the object of increasing interest on the part of experts, decision-makers, and the public in general.

The study of droughts should by no means constitute an end in itself and, fundamentally, it should aim at reducing drought impacts. However, a large majority of the published material on droughts is of a descriptive nature, and looks at droughts from several viewpoints such as agricultural production, urban water supply, environment, or natural disasters. Usually the drought description is presented and in a few cases a partial attempt to identify drought impacts is made. Limited attempts have been made to consider drought research needs or to analyse possible ways of coping with drought impacts by taking adequate measures before, during and after the drought. This is the approach adopted in this paper, following a recent and more comprehensive work in which the author was also involved (Yevjevich, da Cunha and Vlachos (1983)). The purpose of this approach is to define the different ways by which normal water strategies should be modified when drought conditions begin to develop or are already well established.

The prevailing attitude so far has been to respond to drought as a disaster, developing a crisis management strategy which implies a short range approach rather than a risk management long-term strategy which obviously involves the more complicated definition of a set of social options and the consideration of alternative schemes for water resources management.

Particularly in times of crisis, such as the ones in which we are living, societies cannot afford the kind of occasional, partial or late reactions adopted in most drought situations which have taken place recently in many regions of the world. Coherent sets of measures to cope with droughts should in fact ensure signifigant economies and in many cases effectively reduce the dramatic short and long-term consequences of drought.

BASIC DROUGHT CONCEPTS

Drought definitions which have been proposed often depend on the main interest in water use of those who are studying droughts: agriculture, industry, urban areas, power production, navigation, ecology, recreation, etc.

From a meteorological point of view it is usually considered that drought occurs whenever rainfall during a certain period of time is lower than a specified value of precipitation, usually expressed as a percentage of the average precipitation in the region being considered. This meteorological concept of drought is seldom useful for practical purposes as a low value of precipitation (in comparison to the average conditions) may or may not be harmful, depending on the relation between the actual precipitation and the water demand. For example, a precipitation which is 80% of average precipitation can create critical situations in agriculture or water supply in certain regions where water is intensively used, but if a precipitation which is only 20% of the average value occurs over the oceans or a desert area this is not a matter of concern.

A better concept of drought is that of hydrological drought which considers not only the changes in rainfall but also the changes in water discharges both relating to rivers and groundwater.

A more pragmatic drought definition involves not only the water supply, as the meteorological and hydrological definitions do, but also the water demand. Then it is said that drought occurs when in a certain region and during a certain period the water deficit, which is the difference between water demand and water supply, exceeds a certain reference value. In these terms one can say that a drought occurs whenever there is a significant water deficit lasting for a sufficiently long time period and covering a large enough region. The critical values for the water deficit, the extension of the period considered and the size of the region under consideration are usually established in relation to the economic, social and environmental impacts of the drought.

Meteorological drought and aridity are frequently associated, as the more arid regions are those where the variability of precipitation tends to be higher. However, even if both aridity and drought are associated with a lack of water, a clear distinction has to be made between the two concepts. While aridity is a climatic characteristic which corresponds to a steady situation, drought is an extreme phenomenon and can take place both in arid and non-arid regions. Actually it is often in the non-arid regions that the consequences of drought may be more serious since these regions are not usually prepared for the impacts of the drought, which can be very important, particularly the economic impacts.

The possibility of forecasting droughts depends on the understanding of drought mechanisms. The occurrence of droughts is strongly related to atmospheric and oceanic circulation phenomena, but can also depend on other factors such as atmospheric transport of volcanic ashes, dust or other particles causing atmospheric pollution and consequently reducing the solar radiation reaching the Earth. There have also been attempts to correlate drought and the activity of solar spots, but the results so far obtained are not conclusive.

The methods which have been proposed for drought forecasting involve the study of relations between the variations of the values of some hydrometeorological parameters and the occurrence of drought, resorting either to the consideration of deterministic or stochastic modelling.

An adequate forecast of droughts, including the detection of their onset and termination, and consequently the definition of their duration, would be extremely important for an effective implementation of measures for drought mitigation. Unfortunately, long-term forecasting of droughts, including the specification of their duration and intensity (usually defined as the ratio between the total water deficit associated with a certain drought and its duration) is not possible, and reliable forecasts for more than a few weeks ahead are not to be expected.

It is not possible to determine the drought onset when it occurs, as only after the drought has developed is it possible to say if the beginning of a dry period is or is not the actual beginning of a drought.

DROUGHT IMPACTS

Drought impacts are usually classified as economic, social and environmental. Vlachos (1983) has presented, adapted from Yevjevich, Hall and Salas (1978), an exhaustive listing of economic, social and environmental impacts.

Brief comments follow on these three types of drought impacts. One important aspect is the possible occurrence of cumulative impacts, by which impacts of different natures are amplified and/or impacts of successive droughts accumulate, the impacts of subsequent droughts being more serious than they were expected to be if other droughts had not occurred shortly before.

Economic Impacts of Drought

Of the possible impacts of drought, economic impacts are usually the most relevant. From an economic standpoint, droughts may be considered as scarcity crises in a similar sense to energy crises, implying equally an increase in resources and a slowing down of economic activity.

It is essential to try and analyse adequately the costs associated with drought on a regional, national and international scale. From an international point of view, drought impacts on agricultural production can be very important when drought affects large agricultural areas with importance for food production on an international scale. This is, for instance, the case in certain regions of the United States where cereal production corresponds to a significant part of world production.

The drought economic impacts on cultivated lands may also be very important. Examples of these are crop losses or shortages, replacement of agricultural species by other species more resistant to drought but less useful, increase of activity of insects and predators, and the abandoning of agricultural land for other uses. Drought impacts on agriculture may also have indirect impacts on livestock by reducing its quantity and possibly increasing cattle diseases.

The most suitable policies and strategies of water management during non-drought periods may undergo substantial changes when a drought is present. For instance, the criteria and priorities for allocating water to the different users may have to be changed in times of drought. In normal situations it is commonly accepted that urban and rural water supply is considered as a priority use, for several perfectly acceptable reasons, and because water consumption for these types of water supply is only a small part of total consumption. However, in drought situations these priorities can be changed. We may for instance wish to give priority to industrial water supply or to the water supply to certain tourist areas, in prejudice to the urban and rural water supply. This attitude implies public sacrifice and inconvenience which may, however, be less serious for the population than the negative indirect economic repercussions of drastic restrictions imposed on water supply to industrial or tourist activities.

A detailed analysis of the economic impacts of drought is out of the scope of this paper and one can refer to existing literature (e.g. James (1983). Nevertheless, it may be useful to mention in this context that the strategies of water management under drought conditions should be based on the overall expected losses during the whole duration of the drought and not only on what happens during the most critical drought periods, as tends to be done sometimes. Thus it is essential for an effective drought management to try and foresee all drought impacts in as accurate a way as possible.

Social Impacts of Drought

The various types of water users - such as farmers, industrial managers or domestic consumers - are not able by themselves to individually assume the most adequate attitudes in order to reduce drought impact. This makes a study of social impacts of drought very important and also makes it essential that decision-makers provide guidelines on the strategies to be adopted by water users in drought situations.

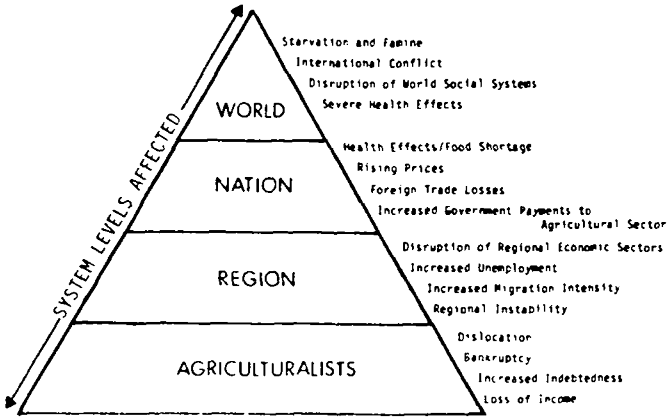
The occurrence of drought forces man to modify his relationship with the environment, by introducing a number of adjustments in his behaviour, the study of which is undertaken by social scientists. Emigration, for instance, has been one of the current forms of social adjustment to drought. Other social consequences of drought are those related to unemployment or deterioration of conditions of hygiene and public health.

Problems of interest for social scientists are the ones related to society's reactions to the announcement of eventual water shortages, water restrictions, or appeals to save water in drought situations. Other problems are those related to the development of a psychological resistance in the face of difficulties associated with drought or with the occurrence of attitudes of solidarity or, on the contrary, hostility when dramatic or merely difficult situations have to be faced.

Individual attitudes towards drought depend on several factors of a personal nature, such as beliefs and/or cultural traditions which may lead to a view of drought as the "will of God" or even as "God's punishment". Another belief that frequently conditions the social behaviour in the presence of drought is the conviction that the next drought will not be as serious as the one which has just occurred. These types of social attitudes may generate difficulties in obtaining public support for the implementation of measures aimed at mitigating drought impacts.

The fact that droughts are by definition non-frequent phenomena explains why the public is usually ill-prepared to deal with them. Decision-makers also tend to react more promptly to more regular problems and infrequent problems like those associated with drought tend to be forgotten until they re-occur.

The range of possible social impacts of droughts has been illustrated by Warrik (1975) in a diagramatic form as shown in the figure on the next page.



Environmental Impacts of Droughts

Environmental impacts of droughts may also be important, ranging from very serious ecological disruptions, as is the case with desertification, to less serious and more localised effects. Examples of this are the reduction of wildlife due to water shortage, the increase of pollution in surface water due to the reduction in volume of water bodies, the increased salinity of groundwater and soil, soil erosion, dust storms, forest fires, plant diseases, insect plagues and increased health hazards.

Droughts are major natural disasasters and together with floods, tropical cyclones and earthquakes, are responsible for more than 90% of the damage caused to man and his environment by natural forces. It may be said, however, that while floods, cyclones and earthquakes are disasters related to extreme events, droughts can be considered almost as non-events. Moreover, earthquakes and cyclones start suddenly, have a comparatively short duration and tend to be localised phenomena, whereas droughts start slowly, have a longer duration and are of a diffuse and insidious nature.

MEASURES FOR DROUGHT IMPACTS MITIGATION

It has been proposed that human responses to extreme events in natural systems can aim at modifying the causes, modifying the losses, or distributing the losses. Glantz (1976) referring to the specific case of droughts, considers that responses to drought can be: tactical responses, when facing a particular drought situation at a particular time; strategic responses, when responses are part of the permanent planning process; and ad hoc responses which are of a spontaneous nature.

Burton, Kates and White (1978) considers four ways of coping with the losses associated with extreme events such as droughts. These four ways - absorption, acceptance, reduction and change - are separated by threshhold levels of awareness, action and intolerance. When there is loss absorption, it means that society absorbs the imapct of the extreme phenomenon and is largely unaware of this being so. When the threshhold of awareness is crossed, then loss acceptance is present, and society adjusts in order to bear the loss, usually by sharing it with a wider group with which the individuals are directly connected. The passing of the action threshhold, implying the setting in motion of specific measures to face the extreme event, leads to a loss reduction attitude. Finally loss change, when the loss is considered as being no longer tolerable, comes when the threshhold of intolerance is reached.

According to the definition proposed by CDWRFA (1977) the measures used to mitigate drought impacts can be of two types:

- proactive measures, which are measures prepared by the systematic actions which may help in the alleviation of drought impacts, i.e., measures which are largely the result of the implementation of a defined planning process;

- reactive measures, which are measures taken once a drought is recognised to be underway, i.e., measures which are largely the result of an improvisation process.

Yevjevich (1983) makes a distinction between measures to be taken before, during and after a drought, which corresponds to the pre-drought, drought and post-drought periods.

The pre-drought period measures are intended to meet the resistance of water users to drought situations. The definition of these measures presupposes the adequate assessment of previous drought impacts, the adequate extrapolation of these impacts for future conditions if and when drought reoccurs, and the evaluation of drought impact mitigation measures including a cost-benefit or cost-effectiveness analysis.

Drought-period measures relate to changes in the pattern of water supply or water demand which may efficiently decrease the drought impacts.

The post-drought period measures are basically intended to minimize drought impacts which have occurred in spite of the measures taken during the on-going drought.

The proactive measures can consist of these three types while the reactive measures relate only to the drought and the post-drought periods.

From a practical point of view the drought mitigation measures can be classified (Yevjevich, Hall and Salas (1978)) into three groups:

- water supply-oriented measures which are intended to increase the available water supply during the drought periods;

- water demand-oriented measures which are intended to decrease the water demand during the drought periods;

- impact-minimization measures which are intended to minimize drought impacts which still occur after the application of the first two types of measures.

Several examples of these three types of measures are explained in da Cunha (1982) and in Yevjevich, da Cunha and Vlachos (1983) and its detailed presentation would be out of the scope of this paper. Thus, only a summary of the measures is given here in sequence. Several of these measures are part of a normal water resources management practice, but only the special or exceptional ways which are resorted to in drought situations are of interest here. In other words, the cost of these measures may not be compensated for by the benefits achieved under normal conditions, but only under water drought conditions.

As supply-oriented measures one can refer to:

- better use of surface water storage capacity
- better use of the groundwater storage capacity
- water transfers between river basins or within the same river basin
- evaporation and evapotranspiration control in reservoirs
- reduction of infiltration losses
- use of soil conservation practices for increasing water yield
- induced melting of snow and ice
- use of a water of lower quality than the one used in normal situations
- temporary relaxation of the water quality standards
- temporary use of water which is not used in normal conditions
- reinforcement of groundwater use
- use of fossil groundwater
- weather modification
- induced accumulation of snow and ice
- interception of dew and fog
- long distance transfer of water
- regional interconnection of water supply networks
- reinforced increase of capacity of water conveyance systems
- water transportation by vehicles or boats
- desalination of salt or brackish waters

As demand-oriented measures one can refer to:

- appeals to the consumers to reduce the use of water
- water reuse
- water recirculation
- establishment of restrictions on water use
- adjustment of patterns of water consumption by users
- economic incentives for reducing water consumption
- regional and local support for water demand-reducing strategies

As drought impact-minimization measures one can refer to:

- drought forecast
- warnings regarding drought situations
- drought insurance
- disaster aid programmes

The measures referred to above have different degrees of effectiveness according to the particular circumstances of each drought situation. It is usually through a judicious combination of some of the measures indicated and through an adequate staggering of these measures that an ideal solution for drought mitigation can be reached.

It is usually essential to consider the joint use of several types of measures related to surface and groundwater, dealing with both water quantity and water quality.

Some of the measures referred to, which are advantageous under certain circumstances, may, in different circumstances, appear as inconvenient, particularly when inadequately associated with other measures. The selection of the more adequate measures to be considered in each case and their staggering in time, should be planned in advance and not left until the last moment when drought is already well developed.

The ideal strategy for mitigation of drought impacts evolves precisely from the selection of the best set of measures to be applied and from the definition of a programme for the application of these measures. To ensure this, it is necessary to develop an information system which at any time allows an evaluation of future drought risks and thus the decisions to be taken on the measures to be applied. The application of the different measures implies, of course, different costs which have to be compared with the benefits (or the reduction of costs) which can be ensured in applying the measures.

Each new drought situation will, to some extent, be different from the previous one. Consequently, it is important that drought impact control strategies should incorporate mechanisms for real-time evaluation of the effectiveness of the applied measures, in such a way as to allow the consideration of necessary corrections to the applied strategies.

For an adequate comparison of the effectiveness of the various measures and the establishment of priorities for their implementation, multiple criteria decision models should be applied. This will provide the decision-makers with a set of possible solutions for progressive and flexible application of the several possible measures.

Drought impact control measures should be applied at different levels, particularly at the levels of the individual, the agricultural or industrial unit, the local or regional authority, the country or group of countries.

Finally, it should be mentioned that public participation in the implementation of drought control strategies is usually critical for the success of these strategies. Often the public is not fully informed on the several possible alternative measures to mitigate drought and this may lead people to think that the measures selected for drought mitigation are not the best ones or that the priorities in applying these measures are not well established.The public should receive regular information through the mass media on the evolution of the drought situation and on the results achieved as a consequence of the application of the selected measures in the different regions suffering from drought.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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APPROCHE NOUVELLE POUR LA MODELISATION DE LA DISPERSION TURBULENTE DE SUBSTANCES SOLUBLES EN RIVIERE

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RESUME

Un modèle stochastique est proposé pour l'étude de la dispersion turbulente en rivière naturelle.

Il permet de restituer la dissymétrie observée sur toutes les courbes de dispersion en faisant intervenir deux paramètres dont l'identification est aisée.

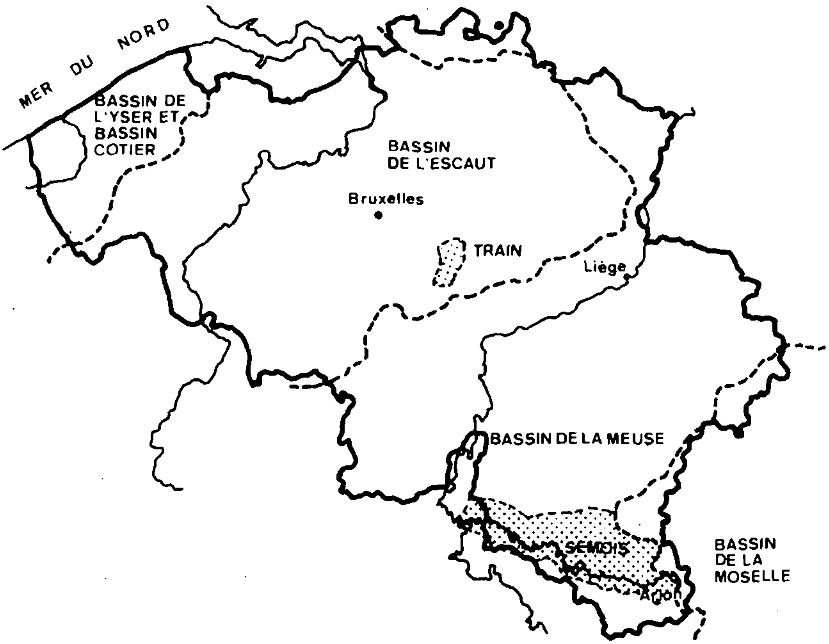
Les performances du modèle sont testées sur des résultats d'essais de traçage. Elles sont nettement supérieures à celles du modèle classique de dispersion basé sur la loi de Fick.

L'introduction de ce modèle est intéressante dans les applications liées à la qualité de l'eau pour lesquelles l'importance du processus de dispersion est fondamentale.

La présente étude a reçu l'appui financier de l'ex Noyau Administratif de l'Eau du Ministère de la Santé Publique et du Ministère de la Région Wallonne.

SYMBOLES

t	:	temps (sec)
v_1 ou v_x	:	composante de la vitesse suivant l'axe x (longitudinal)
v_2 ou v_y	:	composante de la vitesse suivant l'axe y (vertical)
v_3 ou v_z	:	composante de la vitesse suivant l'axe z (transversal)
v	:	vitesse moyenne du courant principal (m/s)
x	:	distance par rapport à la section d'injection (m)
K_{ij}	:	tenseur de dispersion
D_L	:	coefficient de dispersion longitudinale (m^2/min)
$C(t,x)$	:	concentration au temps t à la section d'abscisse x (ppb = 10^{-9} g/l)
W	:	masse injectée (g)
Q	:	débit (l/s)
λ	:	paramètre (10^{-3} min^{-1})
ξ	:	paramètre (10^{-3} m^{-1})



CARTE DE SITUATION DES BASSINS

INTRODUCTION

L'étude du transfert de matières solubles en rivière n'a été abordée que depuis une vingtaine d'années, par suite de l'importance croissante des problèmes de pollution. La plupart des recherches qui ont été entreprises se basent sur une analogie avec la théorie de la diffusion moléculaire, gouvernée par la loi de Fick. Traitée le plus souvent dans le cas monodimensionnel, cette approche classique fait intervenir un paramètre, communément désigné sous le nom de "coefficient de dispersion longitudinale". La méthode, largement appliquée à différentes gammes de débits, ne permet pas, sans artifices, de restituer la dissymétrie spatiale observée sur tous les profils de concentration.

On propose ici une approche différente, de type stochastique,basée sur la description du parcours moyen de la particule d'eau ou de polluant, lié au retard que celle-ci peut accumuler au cours de son déplacement longitudinal dans la rivière. Le modèle, qui fait intervenir deux paramètres, est d'une mise en oeuvre aisée et permet de simuler exactement les courbes de traçage en s'adaptant à leur dissymétrie. L'estimation des paramètres peut se faire simplement par une procédure itérative avec des moyens de calcul très limités.

Le modèle a été appliqué à des séries d'essais de traçage à la Rhodamine WT réalisés sur deux rivières belges, la Semois et le Train. De façon à mettre en évidence son intérêt, on compare les résultats obtenus à ceux fournis par le modèle classique.

TENDANCES ACTUELLES DANS LA MODELISATION DE LA DISPERSION

Bien que la méthode basée sur la loi de Fick soit rarement satisfaisante en rivière naturelle, elle est encore utilisée à l'heure actuelle sous sa forme monodimensionnelle la plus simple. On citera notamment les récents travaux de Brady et Johnson (1981) qui l'appliquent à des essais de traçage en Angleterre sur la rivière Wear.

De nombreux auteurs fidèles à la loi de Fick, mais soucieux de restituer la dissymétrie observée lors des traçages, manipulent ou complètent l'équation fondamentale en fonction de leur interprétation physique du phénomène. Par exemple, Bujon (1983) propose d'introduire dans la formule classique la notion de décroissance linéaire de la vitesse dans le sens transversal, pour arriver à une expression de la réponse impulsionnelle du tronçon considéré dans laquelle interviennent trois paramètres, qu'il identifie par approches successives et qui lui permettent de donner un certain taux de dissymétrie aux courbes simulées. Ces aménagements ne correspondent cependant pas précisément aux caractéristiques physiques du milieu.

D'autre part, plusieurs travaux ont été effectués en vue d'appliquer le modèle classique dans un systeme tridimensionnel. Mais la difficulté introduite n'est pas en rapport avec les resultats escomptés (Fischer, 1970).

Quelques auteurs s'orientent vers des modèles différents. C'est le cas de Beer et Young (1983) qui, en faisant l'hypothèse que le nuage de solution subit tout d'abord une simple translation dans le tronçon étudié suivie d'un passage dans un réservoir, en arrive à un modèle de type (j, n, m), de la forme bien connue :

$$C_k + a_1 C_{k-1} + \ldots + a_n C_{k-n} = b_0 S_{k-j} + b_1 S_{k-j-1} + \ldots + b_m S_{k-j-m}$$

où C_k et S_k sont les concentrations respectivement à la sortie et à l'entrée du tronçon, au temps k
et où les coefficients a_i et b_i peuvent être identifiés par un logiciel d'analyse de séries de données, en choisissant judicieusement les valeurs de j, n et m.

En ce qui concerne la procédure à suivre pour estimer les paramètres des différents modèles, tous les auteurs s'accordent à reconnaître les difficultés d'une estimation à partir des caractéristiques morphologiques de la rivière. La méthode directe par essais de traçage avec, si possible, un traceur conservatif est toujours conseillée.

MODELISATION MATHEMATIQUE DE LA DISPERSION TURBULENTE

La description physique et la modélisation mathématique du mouvement turbulent sont complexes et mal connues. On aborde souvent le problème de la dispersion turbulente en dégageant les effets principaux qui, apparemment, régissent le phénomène. La modélisation est alors réalisée sur les effets eux-mêmes. Cette démarche simplifie considérablement les équations à manipuler mais oblige l'utilisateur à évaluer leurs domaines d'application suivant l'importance de l'effet envisagé.

Théorie classique

Dans la théorie de la dispersion turbulente déduite par analogie avec la diffusion moléculaire pour les mélanges fluides, on suppose que la concentration et la vitesse sont données par la somme d'une valeur moyenne et d'une fluctuation par rapport au temps. Rappelons que l'on représente la covariance "concentration-vitesse" par un terme proportionnel au gradient de concentration. Sous forme tensorielle, l'équation générale, qui décrit le phénomène de dispersion turbulente s'écrit alors :

$$\frac{\partial C}{\partial t} + v_i \frac{\partial C}{\partial x_i} = \frac{\partial}{\partial x_i} [k_{ij} \frac{\partial C}{\partial x_j}]$$

où k est le tenseur de dispersion turbulente.
Dans le cas simple d'un tenseur diagonal et constant, et en ne retenant qu'une seule composante (v_x) de la vitesse (longitudinale), l'équation devient :

$$\frac{\partial C}{\partial t} + v_x \frac{\partial C}{\partial x} = D_L \frac{\partial^2 C}{\partial x^2}$$

où D_L est un paramètre à identifier, appelé communément "coefficient de dispersion longitudinale".

La resolution de cette équation, pour une injection impulsionnelle, s'effectue facilement, soit :

$$C = \frac{W v_x}{Q \sqrt{4\pi D_L t}} e^{-\frac{(x - v_x t)^2}{4 D_L t}}$$

Modèle stochastique

Le modèle stochastique original exposé ci-dessous propose une formulation différente des modèles du type de Fick pour simuler le processus de dispersion dans un cours d'eau.

L'élaboration d'un modèle stochastique implique :

- la discrétisation des dimensions spatiales et temporelles
- la définition du parcours possible d'une particule et les probabilités associées à ce parcours
- le passage à la limite pour tous les incréments de temps et d'espace (vers zéro) ainsi que du nombre des positions possibles (vers l'infini)
- la résolution de l'expression différentielle qui en découle, couplée aux conditions aux limites, et aux conditions initiales du problème envisagé.

On peut démontrer que l'équation générale classique décrite dans le paragraphe précédent est équivalente à un processus stochastique unidimensionnel pour lequel le parcours d'une particule est aléatoire. Les probabilités de transfert sont donc exactement égales, quel que soit le sens de propagation.

L'évolution d'un nuage de colorant injecté ponctuellement et instantanément dans une rivière, ainsi que l'influence des rugosités de fond constatées pour la répartition des vecteurs vitesse sur la surface mouillée d'un cours d'eau, nous ont amené à proposer d'autres probabilités de transfert de la particule dans le cas unidimensionnel.

Dans un contexte stochastique, les hypothèses choisies sont les suivantes :

- le sens du mouvement de la particule est unique
- il existe une probabilité d'arrêt de la particule

$$\xi\, f\,(t/x)\, dx$$

- il existe une probabilité de recirculation de la particule associée à la probabilité d'arrêt

$$\xi\, \lambda\, e^{-\lambda \Delta t}\, f\,(t/x)\, dt\, dx$$

où $f\,(t/x)$ est la fonction densité de probabilité.

Après passage à la limite, l'équation de conservation s'exprime par :

$$\partial_t\, f(t/x) + v\, \partial_x\, f(t/x) = -\, v\, \xi\, f(t/x) + v \xi \lambda \int_{x/v}^{t} e^{-\lambda(t-t')}\, f(t'/x)\, dt'$$

Pour une injection ponctuelle et instantanée g en $x = 0$

$$g(t) = \delta(t) \qquad \text{(distribution de Dirac)}$$

la solution s'écrit :

$$f(t/x) = \lambda Y(t - \tfrac{x}{v})\, e^{-\xi x}\, e^{-\lambda(t - \frac{x}{v})}\; {}_0F_1\,(1/\xi x\, \lambda\, (t - \tfrac{x}{v}))$$

où $Y(t)$ est la fonction de Heaviside
et ${}_0F_1(a/x)$ est la fonction hypergéométrique.

Cette solution peut encore s'écrire en utilisant la fonction mieux connue I_0 : fonction de Bessel modifiée de première espèce d'ordre zéro :

$$f(t/x) = \lambda Y(t - \frac{x}{v})\, e^{-\xi x}\, e^{-\lambda(t - x/v)}\, I_0\left(2\sqrt{\xi\, x\, \lambda(t - \frac{x}{v})}\right)$$

car

$$I_0(x) = {}_0F_1\,(1/\frac{x^2}{4})$$

La distribution de concentration (C) d'une substance injectée en x = 0 pour le cas unidimensionnel, s'écrit :

$$C\ (t,x) = \frac{W}{Q}\, f\ (t/x)$$

Cette solution a été appliquée et testée sur base de mesures expérimentales effectuées en modèle réduit dans un canal vitré à forte rugosité (Michel, 1974).

L'utilisation de ce type de modèle permet de simuler correctement l'évolution de la dispersion dans un cours d'eau, en limitant les paramètres directeurs et en conservant une cohérence entre l'équation de conservation et les hypothèses introduites pour représenter le mécanisme de mélange.

On retrouve, intuitivement, les mêmes principes de circulation des particules dans les adaptations de l'expression classique par rapport à la "Dead zone Dispersion" utilisée dans de nombreux articles (Day, 1975; Thackston, 1970; Valentine and Wood, 1977).

Les retards accumulés par certaines particules de traceur sont, d'après ces auteurs, attribués à des zones tampons interagissant avec le courant principal. Cette propriété est en général introduite par couplage de deux équations contenant un terme de transfert, mais la structure globale de l'équation de Fick est maintenue.

Le modèle proposé dans le présent article peut donc être considéré comme un essai d'unification des hypothèses de base contenues dans l'équation de dispersion Fickienne et dans les adaptations proposées pour les cours d'eau naturels.

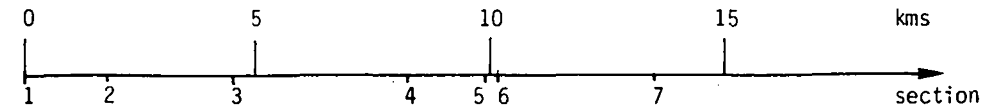
Estimation des paramètres

Les essais de traçage sur lesquels on va se baser ont été réalisés avec un traceur conservatif, la Rhodamine WT, que l'on détecte à l'aide d'un fluorimètre de terrain à flux continu. On dispose de 9 essais sur la Semois et de 4 sur le Train, réalisés à partir d'injections impulsionnelles (voir carte de situation des bassin,à la page des symboles).

L'identification est réalisée par approximations successives. Les données nécessaires sont, outre les données de traçage, le débit moyen du tronçon considéré et la vitesse moyenne que l'on peut estimer à partir du temps de parcours du nuage de traceur.

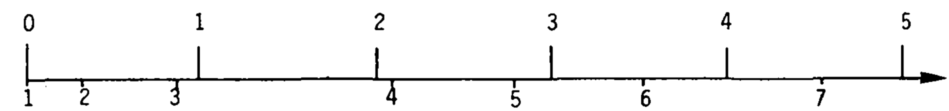
Les résultats sont repris ci-après.

Rivière Semois



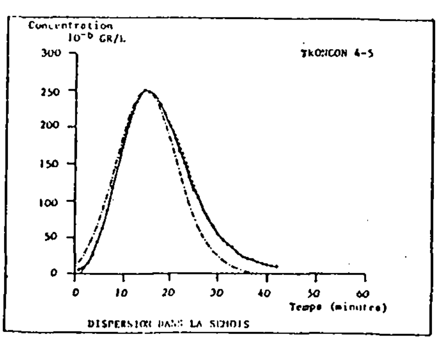
Tronçon	Longueur (m)	Masse injectée (gr)	Débit estimé (L/sec)	Vitesse moyenne (m/sec)	D (m^2/min)	λ ($10^{-3}min^{-1}$)	ξ ($10^{-3}m^{-1}$)
1-4	8310	240	559	0.41	200.8	4.86	1.90
1-5	9830	240	481	0.36	158.3	2.20	0.73
1-8	13490	240	672	0.15	40.9	1.64	0.66
1-9	23790	240	791	0.13	240.5	1.60	1.04
2-3	2770	52	221	0.20	18.2	12.10	4.84
3-4	3710	192	358	0.31	46.3	8.96	2.54
4-5	1520	114	428	0.31	90.7	9.86	5.92
6-8	3350	140	1395	0.25	20.0	8.81	2.64

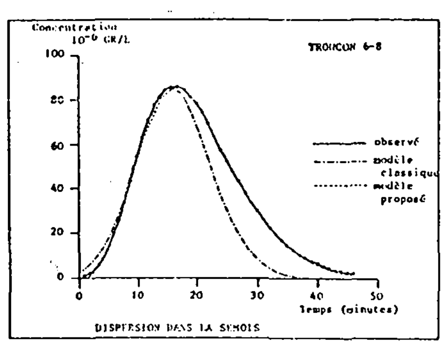
Rivière Train

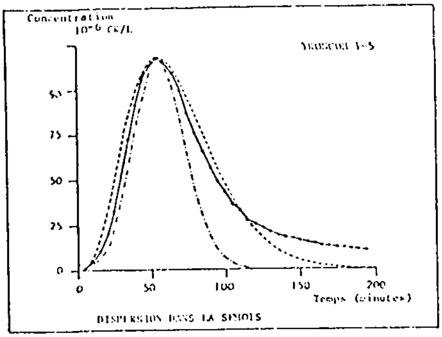


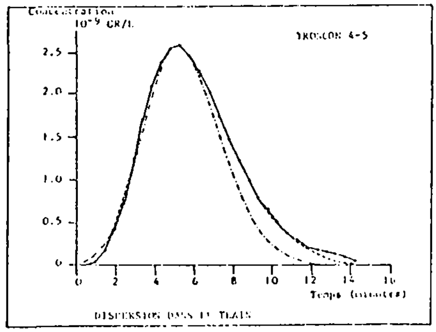
1-2	315	0.7	275	0.34	30.2	24.04	15.40
3-5	1930	2.2	400	0.29	43.4	10.80	4.50
4-5	700	0.6	660	0.44	48.5	31.61	13.65
6-7	1030	1.0	660	0.43	64.5	24.75	8.45

Des exemples graphiques des résultats de simulation sont donnés ci-après.









DISCUSSION DES RESULTATS

- Validité des simulations : les graphiques montrent clairement la supériorité du modèle proposé. Pour des tronçons de longueur moyenne ($\lessapprox$4000 m), on parvient à simuler exactement le phénomène; pour les longueurs supérieures, on observe un décrochement à la fin du nuage, qui est relevé par la plupart des auteurs, quelle que soit la méthode utilisée.
- Facilité d'ajustement du modèle : l'estimation des paramètres est faite par approximations successives, en minimisant la somme des carrés des écarts et en imposant le point de concentration maximale. Une telle méthode est applicable parce que le modèle ne comporte que deux paramètres. Elle se programme facilement sur n'importe quel calculateur.
- Valeurs des paramètres : on constate qu'il existe une corrélation entre les valeurs de λ et ξ, mais elle ne peut être formulée de façon précise. Le rapport λ/ξ se situe, pour les essais, dans l'intervalle (1.6-3.52).

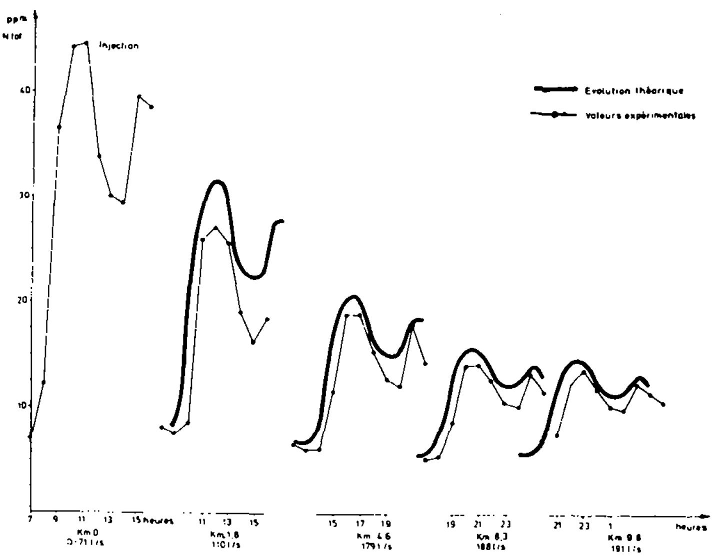
EXEMPLE D'APPLICATION : évaluation de la capacité auto-épuratrice de la Haute Semois

Dans certaines applications, l'importance du processus de dispersion est tout à fait fondamentale. C'est le cas pour les petits cours d'eau où l'on observe souvent une succession de tronçons à géométrie fort variable et une série de petits affluents qui modifient progressivement le débit et diluent les substances polluantes. L'existence de zones mortes, l'importance relative de la végétation des berges ou de massifs de plantes dans les rivières peu profondes peuvent aussi conduire à des coefficients apparents de dispersion longitudinale fort élevées. Si les rejets de polluants présentent un caractère fluctuant, la dispersion longitudinale joue alors un rôle non négligeable dans l'évolution de la concentration de ces polluants.

Dans une première étape, les paramètres du modèle stochastique ont été ajustés à partir d'une série d'injections ponctuelles, comme indiqué précédemment.

Par la suite, l'utilisation du principe de superposition, basé mathématiquement sur l'intégrale de convolution, permet de décrire l'évolution de la distribution d'un polluant pour une source localisée dont les fluctuations quelconques sont connues. Dans cette méthode, on considère le rejet comme une superposition d'injections instantanées dont on peut décrire l'évolution à l'aide du modèle de dispersion défini plus tôt.

Dans le cas de la Semois, on a utilisé les fluctuations de la concentration des polluants dans la rivière, immédiatement en aval de la ville d'Arlon, comme condition aux limites imposées au modèle hydrodynamique. En considérant le cas d'un polluant conservatif, on peut apprécier l'importance relative des processus de dilution liés aux apports latéraux d'eaux non polluées et du processus de dispersion longitudinale. Si on considère par exemple N_{Kj} total comme polluant conservatif dans la zone amont de la rivière où la nitrification est négligeable, on constate que la valeur journalière maximum de cette grandeur est réduite d'environ 50 % par simple dilution et de 30 % par l'effet de la dispersion longitudinale, après un parcours de 10 km (figure ci-après). A la sortie de ce tronçon, la concentration maximum n'est plus que d'environ 30 % de la valeur au voisinage du rejet par l'effet combiné des facteurs hydrodynamiques.
Les écarts avec la courbe expérimentale permettent alors d'estimer la cinétique de production ou de consommation de la substance considérée liée au processus physico-chimique et biologique.



Evolution théorique et expérimentale de l'azote total.

CONCLUSIONS

Le modèle de dispersion proposé est basé sur des concepts physiques nouveaux qui conduisent à une formulation mathématique facilement applicable.
Pour des tronçons de longueur moyenne, le modèle permet de simuler exactement les courbes de traçage observées, ce qui constitue une validation des hypothèses physiques de départ.
L'identification des paramètres est aisée et se programme sur n'importe quel calculateur.
L'évaluation de la capacité auto-épuratrice d'un petit cours d'eau est un exemple qui indique clairement la nécessité d'une connaissance précise du processus de dispersion pour quantifier l'action des facteurs hydrodynamiques sur l'évolution des paramètres de la qualité des eaux.

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ENVIRONMENTALLY SOUND MANAGEMENT OF FRESHWATER RESOURCES

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ABSTRACT

The main principles of environmentally sound management of water resources over the river basin are outlined, information on UNEP comprehensive water programme for this type of management is presented and the improvement of integrated river basin development and its decision making is contributed by this paper. The main functions of freshwater bodies are natural resources, ecosystem and landscape formulator. Evaluation of the present practice of water management is mainly oriented to the natural resource function. There is a need to consider all of the functions by an integrated approach, which can be implemented by environmentally sound management of water resources in the river basin as a whole. It makes it possible to harmonize continuously the different interests of socio-economic development and water-related (natural and man-made) environment over the entire river basin. It applies at the project as well as the basin-wide level and takes into account the co-existing three functions of water bodies. The output of such management will be a harmonized water-related environment which is good (not only acceptable) from socio-economic and ecological viewpoints throughout the process of long-range river basin development. Criteria and evaluation of environmentally sound management of river basins. Procedure to develop this management. UNEP's programme. Development of Action Plans and world-wide system of pilot river basins. Co-operation with the Governments, UN-agencies and non-governmental organizations.

Keywords: water resources, environment, river basin, environmentally sound development, harmonization, socio-economic interests, conflicts, integrated approach, criteria, state of environment, UNEP water programme.

1. INTRODUCTION

There is no doubt that since the Stockholm and the Mar del Plata Conferences, there have been considerable changes in our understanding and perception of environmental issues related to water and land resources. Previously land, water and other natural resources had been exploited without restraint, and wastes discharged freely into water, soil and air, without responsibility. The 1970s brought into focus the general realization that the different physical components of the environment have limited assimilative and carrying capacities and that pollution control measures must be instituted to safeguard the environment and the quality of human life. More important has been the growing realization that the natural environmental resources of water, soil, plants and animals constitute the natural capital on which man depends to satisfy his needs and achieve his aspirations for development. The wise management of these resources has demanded positive and realistic planning. This balances human needs against the potential which the environment has for meeting them.

There is accordingly in UNEP today a search for integrated physical, socio-economic and environmental planning, a search for strategies that will provide environmentally sound development. It means a progress in which for example we are not looking for 100% clean water, or leaving the national resources completely untouched. We are talking in terms of trade-offs and mixes and we are trying to find best compromises. For this development are not short-term one-time projects but growth strategies which can be sustained over a long period of time - which means that the availability of natural resources, both renewable and non-renewable, and the regenerative capacities of the environment are kept centrally in mind, permitting that kind of development.

A clear and comprehensive appreciation of the nature of environmental problems leads to an understanding of the interaction between man and nature on the one hand, and the design of practical development policies and objectives on the other. Interactions among forces of change, namely resources, environment, people and development, make it necessary for decision makers everywhere to think in terms of trade-offs between alternative courses of action. Each possible development objective must be carefully evaluated in order to identify its environmental consequences. The recognition of the critical relationship between environment and development leads us directly to the need for new and more integrated approaches to environmental management (Tolba, 1984). A common philosophy must underlie our management efforts, which we would sum up as follows:

1. Economic and social development must be pursued to meet the basic human needs of all people, and to secure a better future for them;

2. Environmental processes must be thoroughly and widely understood;

3. The productive capacity of the environment must be maintained and resources used rationally.

Rational management of the environment, which means essentially making the best use of nature's resources to meet basic human needs without destroying the ecological basis on which sustained, balanced development depends, calls for a new approach to solving rapidly multiplying environmental problems. This new inter-disciplinary and cross-sectoral approach must be based on sound knowledge of the interacting elements within the larger frame of development, which makes possible the choice of alternative, environmentally sound development strategies.

In order to promote adoption of this approach to the management of freshwater resources, the purpose of this paper is to outline the main principles and activities of environmentally sound management of water resources over river basins, to present information on UNEP comprehensive water programme for this type of management and to contribute to the improvement of integrated river basin development and its decision making. To meet this purpose, the paper is organized in the following way. In the next section the challenges of socio-economic development in river basins are discussed as a background. This is followed by the description of environmentally sound river basin management and UNEP comprehensive water programme. Finally, conclusions and recommendations are summarized.

2. CHALLENGES OF SOCIO-ECONOMIC DEVELOPMENT IN RIVER BASINS

The natural availability of water resources in a river basin, generally, does not coincide with the growing water requirements of the socio-economic development and the constraints of environmental management in the same basin. Consequently, there is a need to establish a continuous balance among natural supplies, water demands and environmental requirements over space, time, quality, quantity, and energy. This balance can be established and maintained by river basin development which is an increasingly integrated, planned and comprehensive long-term process. Its purpose is to achieve the most efficient use of water resources on a basin-wide scale during socio-economic development. The criteria for most efficient use basically will depend on the constraints of the socio-economic growth and environmental management.

The development process of river basins from the point of view of water management can be divided into three consecutive periods: the natural (period I), the developing (II) and the mature or fully developed (III) phases of river basins (Dávid, 1976).

In period I, there is no significant human interference in the river basin, the water resources are in conformity with natural conditions. The water projects have single purpose, and their capacity is not significant in comparison with the natural supply of water. In period II, deliberate human interference is restricted to that of a local and regional character and grows step-by-step to basin-wide dimensions. Under its influence, the natural runoff system gradually becomes more regulated. Multipurpose integrated water projects and their growing systems are constructed with increasingly larger capacities. The storage space and the role of water transfers (import, export, inter-basin transfer, etc.) increase. The usable water resources, the importance of water demand control, the amount of sewage effluent, the number of point and non-point pollution sources, and the extent of treatment also increase. The deterioration of the water quality has been started and it induces the development of water quality control. Finally, in period III the redistribution of completely regulated water resources among users and the prevention of water damage, etc. are continuously undertaken by the fully developed, basin-wide and controlled multipurpose water resources system. In this redistribution, the supply to meet the fresh-water demand, water purification and effluent treatment form a unified system being in dynamic equilibrium with regard to the socio-economic and environmental conditions. Further development is based on water demand control and large inter-basin transfers.

Considering the outlined river basin process it is concluded that the objectives, the means, the changes of the predominant activities, the interconnections among activities, the importance of the water projects and its changes during the life-time of the projects, the scenario of the river basin's water management structure depend on the stage of river basin development process. This stage can be measured by development function described by Dávid et al (1979). As a consequence of this the water resources management and development and the associated environmental impacts, the freshwater ecosystems are becoming more and more complex in their structural, spatial and temporal dimensions during this development.

The perception of functions of freshwater bodies (rivers, lakes, aquifers, etc.) are changes during river basin development. Freshwater is not only a renewable natural resource for which no substitute exists, but forms a very important part of ecosystems and landscapes. Contrary to the present practice, which takes into account mainly its function as a natural resource, all these functions should be considered simultaneously. The interactions between environmental conditions, the activities of man and freshwater ecosystems are becoming more complex, and the water system of the river basin as an environmental unit and a unit of development is changing very rapidly. This means that the environmental impacts and their management cannot be viewed only in the context of specific water and water-related projects, but also should be considered on a basin-wide scale. This challenge is especially important in the case of international water systems.

To follow this approach some changes in the present practice of environmental assessment and impact studies of water projects are also needed. Firstly, we need not only an impact study but also an action plan for impact management which considers both short-term and long-term dimensions. Secondly, there is a need not only to include such management studies into the decision-making process, but also to incorporate them as integral parts of it. Present experience demonstrate that there is a great difference between inclusion and true integration. Management plans and actions which include assessment and are based on a systems analysis approach can help to promote the latter.

Solving the "environment versus socio-economic development" conflicts generated in river basins by water projects and water-related activities (e.g. open-pit mining projects, road construction, forest management activities, etc.) requires a new comprehensive basin-wide approach which reconciles the different interests and integrates them into environmentally sound development. There is an increasingly urgent need to put the principles of environmentally sound management of water resources into practice. All of these can be achieved through the environmentally sound management of water resources over the entire river basin, with the river basin considered as the basic spatial unit.

3. THE PRINCIPLES AND ACTIVITIES OF ENVIRONMENTALLY SOUND MANAGEMENT OF WATER RESOURCES

The environmentally sound management of water resources is regarded as a management process which makes it possible to harmonize continously the different interests of water related socio-economic development and the water-related (natural and man-made) environment over entire river basins or ground-water systems, and thereby accelerate regional development and provide improved living conditions.

It applies at the project level as well as the basin-wide level and takes into account the co-existing natural resource and ecosystem-landscape functions of inland waters. It includes the environmental management of surface and ground water, in terms both of quantity and quality and of the dynamic interrelationships between water, land, energy, climate and biota.

It incorporates environmental considertions into a wide variety of water management and development activities connected with water and water-related projects in a water system, ranging from assessment, monitoring, policy making, planning and analysis of decisions, through design, and construction, to operation, maintenance and rehabilitation. It promotes the practical application of the principles of environmentally sound resource development in the water management.

The output of this management process is a harmonious river basin development and water-related environment (e.g. good-quality surface and ground-water sources, limited or no erosion, no water shortages or flood damage, etc.) which is not only acceptable but good from the social and ecological viewpoint throughout the process of long-range development in the river basin.

The environmentally sound management of water resources should involve a set of activities implemented over the whole river basin. The check list of the main activities can be summarized as follows:

i) Development and operation of a monitoring, assessment and surveying system for the water-related environment, especially for water quality and quantity in the water system and its watershed, with special reference to the environmental impacts of large water projects. It should be developed gradually in harmony with the requirements of the river basin development process. Land use patterns and ground-water resources should be monitored too. The use of remote sensing would be desirable. This system should contain provision for forecasting environment-related processes which are capable of reaching extreme values, such as run-off, floods, sedimentation, droughts, water shortages and variations in water quality, soil moisture, water demands.

ii) Implementation of a comprehensive and effective water quality management programme of surface and groundwater resources which will include:

- the control of point and non-point sources;
- proper water and sewage treatment;

- safe waste and treated water disposal;

- proper water supply and canalization;

- improved water-use technology, including waste water reuse and recycling, etc.;

- basin-wide run-off regulation.

The implementation of this programme will require:

- The harmonized development of the safe drinking water supply and sewage canalization and treatment in order to improve the health conditions of the population and to avoid the degradation of ground-water resources and water quality along shorelines;

- The environmentally sound development of community and industrial water management. Polluting substances must be controlled at the source, and it must be the responsibility of the users of these substances to ensure that they are not discharged in the environment where the damage they cause may be irremediable;

- The environmentally safe disposal of harmful industrial liquid waste and both industrial and domestic solid waste in surface and ground-water resources;

- Effective control of water levels and discharges in the water system by reconciling the interests of ecosystem management, different water users (industry, irrigation, drinking water supply, etc.), fisheries, recreation and navigation. Effective flood plain management and flood warning in both rural and urbanized areas;

- Effective control of the use and handling of herbicides and chemical fertilizers in agriculture;

- Controlled development of urbanization, recreation and industrialization in the basin, bearing in mind the absorptive capacity of the water and related environment;

- Co-ordination of economic activities in the basin in keeping with the ability of the water and water-related ecosystem to tolerate these activities.

This activity should make it possible to maintain suitable water quality and quantity in both surface and ground water, and to ensure safe conditions for drinking water supply and sanitation and to meet other water requirements and to avoid water shortages.

iii) Implementation of a <u>conservation</u> programme for water-related <u>fauna and flora</u>, both in the water system and in the basin as a whole, to avoid drastic changes. The preservation and development of national parks and reserves.

iv) Implementation of <u>soil conservation and erosion and sedimentation control</u> programmes over the whole basin, including the applictation of proper agricultural technology.

v) Implementation of combined forest and watershed management to conserve the water on the watershed.

vi) Effective water-related and forest-related land use planning and landscape management, including the rehabilitation of critically eroded or degraded areas, e.g. open mining pits, etc.

vii) Assessment and development of the water related social environment and human impacts, especially health aspects of water development.

viii) Implementation of a comprehensive and integrated river basin planning process. Multicriteria planning procedures and systems planning methodology are suggested, including cost-effectiveness analyses. Through these, alternative development strategies can be simulated, their short-term and long-term impacts evaluated and best compromise solutions can be developed.

ix) Establishment and operation of a suitable legal and institutional system with adequate powers involving the related national, regional and local government authorities and organizations, and representatives of the public, to co-ordinate and control the different technical, legal, financial and social activities involved in environmentally sound management of the water system.

x) Development and operation of comprehensive interdisciplinary research and training to provide continuous support for the environmentally sound management of the water system.

xi) Implementation of a public information programme on, and public participation in, decision-making and control of this type of water management, bearing in mind that genuine public participation is a very important prerequisite for the success of water management.

Considering the above main activities it can be stated that if they are properly implemented the water resources of the river basin are managed by an environmentally sound way. The actual activities can be selected from the above check list according to the circumstance in the river basins. The set of actions needed for environmentally sound management and the system of criteria to measure their effectiveness and the state of environmentally sound management are different for each of the river basins. They can be determined by a detailed analyses. Considering the wide variety of the possible actions and the limited resources available, it is advisable to be highly selective in the choice of the necessary activities. The activities have different priorities. The harmonization of the different socio-economic and environment interests can be planned and the co-ordination of the listed main activities can be developed by the integrated river basin planning process, which can give inputs to most of the other activities. The selected activities of a certain river basin can be incorporated into an action plan. The Action Plan of this river basin includes all of the activities which are needed in this basin to develop the environmentally sound management of the water resources.

The development of environmentally sound management of water resources in a river basin can be implemented in three steps as follows:

Step I. Preparation of a draft action plan for environmentally sound management of the water resources management of the river basin to define impacts, goals and specific water resources environmental management policies and activities.

Step II. Adoption of the action plan, including the establishment of legal and institutional machinery for its implementation. This will involve cooperations between the interested agencies, institutions, countries, etc.

Step III. Implementation of the action plan, including regular reviews of progress.

This procedure can be applied in the national and international river basins. It ensures the incorporation of environmental aspects into the management and development of inland water resources. It would be useful to develop the environmentally sound water resources management in the major river basins of the world in the next decades.

4. UNEP'S COMPREHENSIVE WATER PROGRAMME

As water is one of the main elements of the environment, both at the global and the national scale, UNEP has a concrete mandate in this field. Water management was one of the initial main activities of UNEP. During the last twelve years a series of water activities initially in form of independent projects, was implemented. They dealt with specific elements of environmental impact assessment of water management systems and environmentally sound management of the water resources. However, other activities of UNEP such as the Global Environment Monitoring System (GEMS), combat against desertification, soils, health, human settlements, regional seas and other programmes, have also important water components.

The water projects of UNEP have been implemented in different parts of the world. Considering that UNEP is not an executing agency, these projects were implemented by and in very close co-operation with the UN Regional Commissions, UN-DTCD, UNESCO, WHO, FAO, UNICEF, WMO, and others. The results of these activities have been translated into governmental decisions, various guidelines and publications, and generally improved the expertise of water experts. Furthermore they have contributed to integrate environmental assessment and impact studies in the planning and decision making processes for water projects, especially drinking water supply, sanitation and irrigation.

In the light of these circumstances, and considering UNEP's philosophy on environmentally sound development, we in UNEP are now working on the preparation of a new comprehensive water programme for the Environmentally sound management of inland waters (EMINWA).

This programme (UNEP, 1984), is designed to integrate environmental considerations to management and developement of inland water systems (rivers, lakes, groundwater aquifers) with a view to reconciling conflicting interests and ensuring regional development of water resources in harmony with the water-related (natural and man-made) environment over entire river basins and groundwater systems. It is planned to contribute and promote the environmentally sound management of water resources in the river basins, especially in the international river basins according to section 3.

This long-term programme is to be implemented by means of an integrated system of specific short-term inland water projects (in rivers, lakes, aquifers, large water projects, their systems and basis, etc.), and supporting projects for training, institution-building, demonstration, etc. The inland water projects should be national and international. Priority is given by UNEP to the international water systems. The main activities of the programme are: (a) development of environmentally sound management in more and more inland water systems, (b) preparation of a manual of UNEP principles and guidelines on environmentally sound management of inland water, (c) use of inland water projects for demonstration purposes, and development of a world-wide system of pilot river basins; (d) training and institution-building; (e) co-operation with governments, United Nations organizations and non-governmental organizations; (f) a regular world-wide assessment of the state of the environment in inland water systems; and (g) public information activities. The development of environmentally sound management of water resources in a river basin involves the three steps mentioned in the previous section.

The first inland water project proposed is a project on environmental management of the common river system of the Zambezi. The inland water projects in the different regions could constitute a system of regional sub-programmes. It is suggested that the African EMINWA should be given priority in view of the serious drought in that continent.

EMINWA is initiated and co-ordinated by UNEP and implemented in co-operation with the organizations of the UN system. It may be considered as a UNEP contribution to the follow-up of the Stockholm and Mar del Plata Action Plans, the programme for the International Drinking Water Supply and Sanitation Decade and the International Hydrological Programme. It will link the regional seas programme of UNEP to activities in river basins feeding into them.

The work plan of the programme has two phases for the medium-term period of UNEP activities: a preparatory phase in the first biennium (1984-85) and an implementation phase in the second and the third biennium (1986-89).

It is hoped that this programme will contribute to promote the integrated river basin approach as one of the main message of the Mar del Plata Conference and to improve the state of environment, both globally and regionally, requested by the Stockholm Conference.

5. CONCLUSIONS AND RECOMMENDATIONS

Considering that there is a global need for environmentally sound development in general and that the challenges of socio-economic development in river basins also require the harmonization of socio-economic and environmental interests in particular, the principles and main activities of environmentally sound management of freshwater resources in the river basins are outlined here. To promote this type of management a comprehensive water programme of UNEP has been suggested. All of these have led to draw the following main conclusions and recommendations:

1. The environmentally sound management of water resources makes it possible to harmonize continously the different interests of water related socio-economic development and the water-related (natural and man-made) environment over entire river basins or ground water systems. It takes into account the co-existing natural resource and ecosystem-landscape functions of water resources.

2. This type of management is able to contribute and improve the integrated river basin development by incorporating the environmental considerations into a wide variety of water management and development activities connected with water and water related projects. It promotes the practical application of the principles of environmentally sound resource development in the water management.

3. It is recommended to develop the environmentally sound management of water resources in more and more river basins of the world.

4. It is recommended that the Governments co-operate with UNEP and other UN organizations to promote the application of environmentally sound water resources management.

5. A world wide system of pilot river basins with environmentally sound water resources management is suggested to be developed.

6. It is recommended that the International Water Resources Association join to EMINWA programme and cooperate with UNEP to implement it.

It is hoped that the above recommendations will be discussed by the Congress and can be considered to contribute to the final recommendations of the Vth World Congress on Water Resources.

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A MATHEMATICAL MODEL TO FORECAST COMMUNITY WATER DEMAND

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ABSTRACT

A mathematical model to forecast community water demand is developed from the analysis of past water consumption and factors which are related to the community water use patterns.

Total population of the community, number of water supplied population, population density, community products, water rate and other essential elements which give great influence to water consumption are considered as explanatory variables for community water demand and are determined as the factors of water demand through the factor analysis.

Those factors are then analyzed and correlated with the dependent variables by multiple regression analysis. Total gross and per capita community water demands are considered as dependent variables for which forecasting demand model is to be determined.

Finally, the projection of water demand is made from this relationship and compared with the classical or conventional estimation of community total gross and per capita water demand.

Among the developed forecasting models for total gross community water demand, linear-1 model indicates more adequacy for expected quantity and others are considered irrelevant for the forecasting purpose. For per capita community water demand, models of linear-2 and log-linear-2 are more significant for the forecasting purpose. In the general view, models for per capita water use are more adequate than those for total gross water use.

INTRODUCTION

Water demand forecasting is very important and essential for long-term plan of water supply project and its management policy for a community. As the basic conception of water resources is shifting from free good to economic good, the intensity and efficiency of water resources management must be developed further. Water consumption by a community is influenced by many factors, such as population, water price, community cultural behavior, industrial components, commerical and social use and climatic situations. As predicting future situations of these elements is very difficult, forecasting the community future water demand is uncertain.

The historical or conventional approach to forecasting demand water is as follows(Takeuchi and Takahasi, 1971). The future water demand for a community is estimated over a specified time horizon based on an average water consumption per capita and per unit industrial production. The total demand of community is then calculated by multiplying the population projection by the estimated average consumption rates. This conventional approach is relatively simple but it ignores the relationship between investments, costs, prices and the quantity of water demand. Such forecasts implicitly assume that the technical, economic and behavioral characteristics of the community are stable, an assumption that is demonstratably incorrect.

As a result, many investigators for community water demand have taken issue with various approximations implicit in the procedure. A principal difficulty associated with better forecasting and thus better planning, however, is not the lack of recognition on the part of current planners of the shortcomings of the historical procedure, but instead with the availability of data upon which to base better decisions. Some of the problems associated with historical or conventional water demand forecasting for a community is discussed more fully as follows.

A criticism of the historical forecasting procedures is that they are trying to solve a very complex problems with too simple solution procedure. A useful generalization of the historical procedures is that they assume that demands are requirements. The fundamental misconception in the requirements approach is that the 'water is different' philosophy(Milliman, 1963) i. e., water is unique and should not be regarded as an economic good. As discussed above, the conventional approach to forecasting demand water employed many factors that could affect the future demand. But in many cases in this study community, Taegu city, as well as many parts of the country estimation of future demand are being made by the conventional method which is relatively simple but with uncertainty. The total demand was then calculated by multiplying population projection by the estimated per capita water use from the data base of past consumption.

This water demand study concerns the city of Taegu situated in the southern part of Korea as shown in Figure 1. Many factors affecting demand function are collected and analyzed for determination of future demand models. Reasonably good forecast can be made with derived models by projected explanatory variables.

PATTERN OF WATER CONSUMPTION IN TAEGU

The area of the city of Taegu is almost 180km^2 in 1980. The population is 1,600,000 almost 4 percent of the country. Population growth rate is 3 to 6 percent average per year from 1960. People connected to the municipal water system is about 48 and 95 percent in 1960 and 1980 respectively. Average per capita consumption is 102 liters and 223 liters in 1960 and 1980 respectively.

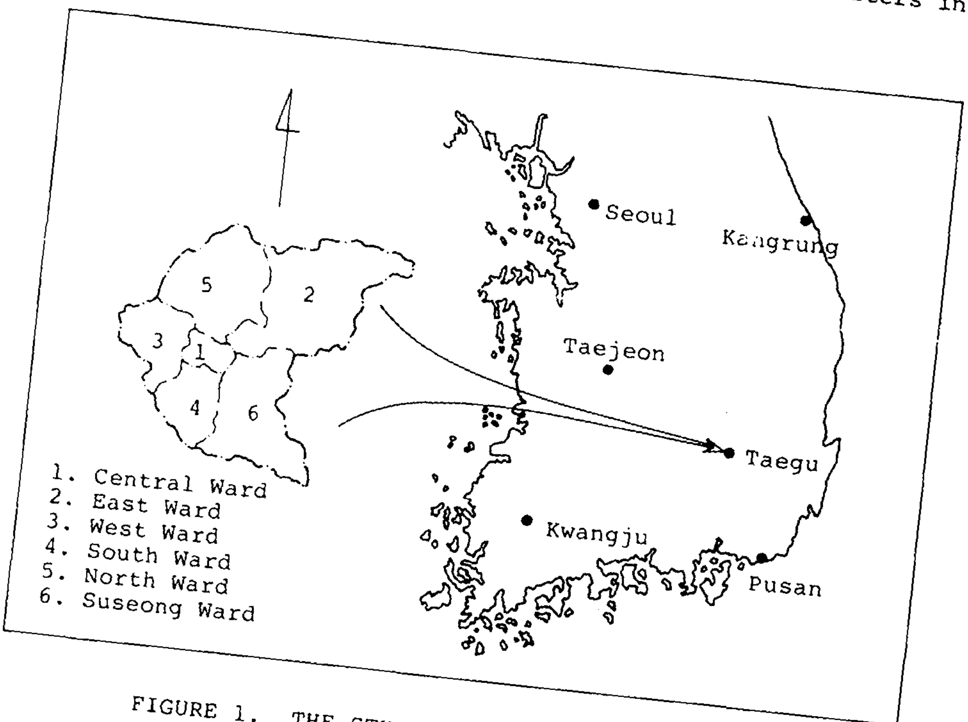

FIGURE 1. THE STUDY AREA-TAEGU, KOREA.

DATA ASSEMBLY AND ANALYSIS

The water consumtion data to be analyzed must be the result of consumption without constraint to induce better forecasting models. Thus the water consumption data in 1980 which was not constrained are used for factors analysis. As the first step in the analysis of community water demand, all the variables which are considered of potential value in explaining the community water demand are enumerated. Various activities in a community contribute to the observed overall water use pattern of the community each of these activities has to be represented by appropriate explanatory variables(Hashimoto, T. and L. de Maré 1980). Data for the year 1980, where requirement constraint was not imposed, are available for the explanatory variables, i. e., urban population, population served, percentage of served population to total population, served area, percentage of served area to total area, population density, per capita income, water rate, number of commercial business, number of industrial employees. Table 1 shows the data for the year of 1980 which are available for the variables and 1980 is the most recent year for which extensive data are available. In the table, x_1, x_2 ..... x_n are explanatory variables, and Y and y are dependent variables for total and per capita use respectively.

From the data base assembled above, the appropriate explanatory variables must be indentified for each dependent variables. Statistically, there are several properties of explanatory variables to be considered in this identifying process(Hashimoto, T. and L. de more, 1980). The following two properties are generally accepted as important for explanatory variables:
1) high correlationship with the dependent variables.
2) low correlationship with other explanatory variables.
When there are multiple candidates which have similar characteristics and satisfy these properties in the same degree, and additional property to be considered is;
3) high relative variation in the observed values of the variable. The relative variation can be measured by the coefficient of variation defined as the ratio between the standard deviation and the mean. If the model is to be used for forecasting, another important property of explanatory variables is;
4) ease of reliable forecasting

TABLE 1. DATA ON EXPLANATORY AND DEPENDENT VARIABLES FOR THE YEAR 1980.

Explanatory Variables	The Study Community(Wards) Central	East	West	South	North	Suseong
x_1 = total population in 10^3 persons	219.29	249.32	392.88	267.84	273.37	204.77
x_2 = population served in 10^3 persons	200.60	216.20	380.04	267.20	247.81	183.66
x_3 = ratio of pop. served in %	91.48	86.72	97.80	96.76	90.65	89.69
x_4 = supplied area in Km^2	7.01	17.74	19.50	8.35	17.87	25.92
x_5 = ratio of sup. area in %	100.00	30.80	91.29	48.13	46.71	68.00
x_6 = population density in 10^3 person/Km^2	31.28	4.33	18.39	15.42	7.15	5.37
x_7 = per capita income in 10^3 Won	859.10	610.60	464.70	771.70	910.70	861.70
x_8 = water rate in Won	108.25	80.97	84.97	79.06	74.18	81.61
x_9 = commercial business in 10^3 unit	7.74	3.77	6.35	5.13	5.06	3.04
x_{10}= industrial employees in 10^3 persons	0.31	3.04	26.17	0.19	56.47	3.80
Y = total community use in 10^3 m^3/day	29.63	25.27	46.72	31.06	50.21	24.11
y = per capita use in l/day	147.54	116.83	122.86	116.19	202.54	138.34

These 4 properties can be used as criteria for selecting appropriate explanatory variables at different stages of the analysis. Table 2 shows the result of principal factor analysis according to above mentioned process. Each factor can be interpreted by looking at those variables having high(positive or negative) loadings in the respective factor. Three principal factors were identified by factor analysis. The first factors may be interpreted as representing the level of income, water rate and industrial and commercial activities. Total population, population served and the percentage of population served to total population have high loadings in the second factor. The second factor may then be interpreted as representing the level of urbanization. The third factor has high loadings variables of x_5 and x_6(percentage of supplied area to total area and population density respectively). It can be also interpreted as urbanization as the second factor.

x_1 respresenting total population, is strongly related to the size of community as seen from the high loadings as asociated with population served and supplied percentage. The third factors are close relationship with the second factors explaining the level of urbanization. Thus for identification of equations for the community water demand, the following six variables which are considered potentially very important in explaining the dependent variables are selected. Those factors are population served(x_2), ratio of population served to total population(x_3), per capita income(x_7), water rate(x_8), commercial business(x_9) and industrial employees(x_{10}), excluding x_1, x_4, x_5 and x_6.

TABLE 2. FACTOR MATRIX(CORRESPONDING TO THE DATA BASE GIVEN IN TABLE. 1).

Explanatory Variables	Principal Factors 1	2	3
x_1 = total population in 10^3 persons	0.070	0.975	0.026
x_2 = population served in 10^3 persons	-0.059	0.986	-0.062
x_3 = ratio of pop. served in %	-0.508	0.781	-0.292
x_4 = supplied area in Km^2	0.681	0.157	-0.562
x_5 = ratio of sup. area in %	0.185	0.258	-0.873
x_6 = population density in 10^3 person/Km^2	0.302	0.040	-0.949
x_7 = per capita income in 10^3 Won	-0.979	0.086	0.174
x_8 = water rate in Won	-0.979	0.087	0.174
x_9 = commercial business in 10^3 unit	-0.979	0.087	0.174
x_{10}= industrial employees in 10^3 persons	-0.979	0.087	0.175
Cumulative Percentage of Eigenvalue	0.492	0.782	0.953

DETERMINATION OF WATER DEMAND MODELS

The selection of appropriate explanatory variables is closely connected with determination of model structure for each dependent variables. From previous factor analysis, six variables were identified as potentially important in explaining the both total gross and per capita demands. Both total gross and per capita demands are the dependent variables, and linear and log-linear forms are considered in this study. The effects of explanatory variables on the dependent variables are assumed to be either additive or multiplicative(Hashimoto, T. and L. de Marè 1980).

Linear model is explained as; Log-linear equations;

$$Y = a_0+a_1x_1+a_2x_2+ \cdots +a_nx_n \qquad Y = a_0 \cdot x_1^{a1} \cdots x_n^{an}$$

Where Y: dependent variable

$x_1, x_2 \cdots x_n$: explanatory variables

$a_0, a_1, a_2, \cdots a_n$: regression coefficient

Stepwise regression was run on previously selected six variables with the above linear and log-linear forms assumed to be an appropriate model structure for the total gross and per capita water demands. The results are summarized in table 3(a) and 3(b) with t-value(verifying regression coefficient), R^2(multiple correlation coefficient) and F-value.

The result shows that the industrial employees and ratio of population to total population are the most important variables in explaining the total gross water demand. For per capita water demand, per capita income and industrial employees are the most important variables. Commercial business water rate are also very high relationship with total gross and per capita water demands.

PARAMETER FORECAST FOR EXPLANATORY VARIABLES

Official data of water works in Taegu for 21 years(1960-1980) were collected for long-term parameter forecasting of explanatory variables to be used for projection of future water demand by developed forecasting models. The trend of past data was identified to conform with either equation of linear curve, secondary curve, exponential and modified exponential curves and logistic curve, i. e.,

$$x_i = a + bt_i \ , \quad x_i = a+bt_i+ct_i^2 \ , \quad x_i=a \cdot b^{t_i} \ , \ x_i=k \cdot a \cdot b^{t_i} , \ x_i=\frac{1}{k+a \cdot b^{t_i}}$$

Where x_i: model variables(explanatory variables)

t_i: estimation year

a,b,c,k: regression coefficient.

Regression coefficient were developed and verified by Chi. sq.-test and residual percentage. Applying determined equations with the data base, model parameters forecasting was made for 30 years up to 2010;

FORECASTING COMMUNITY WATER DEMAND.

As discussed imprevious section, the industrial component represented by the number of employees was found to be most important in explaining water demand. Next important components identified are ratio of population served to total population and per capita income for total gross demand and per capita

TABLE 3(a). STEPWISE REGRESSION OF TOTAL GROSS COMMUNITY WATER DEMAND.

Explanatory Variables	Regression Coefficient [a] (t-value) Linear Model 1	2	3	4	Log-linear Model 1	2	3	4
X_2				0.036				-0.042
				(6.60)+				(0.05)
X_3		1.022	0.816	0.341		3.814	2.134	2.316
		(3.58)*	(3.15)+	(3.77)		(1.93)	(1.15)	(0.48)
X_9			1.081	1.160			0.425	0.428
			(1.68)	(8.46)+			(1.66)	(1.15)
X_{10}	0.446	0.432	0.430	0.394	0.089	0.095	0.103	0.105
	(4.16)*	(7.99)**	(10.07)**	(37.33)*	(1.73)	(2.39)+	(3.22)+	(1.56)
Intercept	27.810	-66.220	-52.770	-17.830	29.880	9.58×10^{-7}	0.001	0.001
R^2	0.901	0.982	0.993	1.000	0.653	0.863	0.945	0.945
F	17.31	40.63	44.47	754.70	2.98	4.38	5.56	2.09

a) The first step column, for instance, reads as follow: $Y=27.810+0.446x_{10}$, $Y=29.880x_{10}^{0.089}$.
b) The double and single asterisks and the cross denote the significance at 1%, 5%, and 10% levels respectively.

TABLE 3(b). STEPWISE REGRESSION OF PER CAPITA COMMUNITY WATER DEMAND.

Explanatory Variables	Regression Coefficient [a] (t-value) Linear Model 1	2	3	4	Log-linear Model 1	2	3	4
x_2				-0.116				0.294
				(245.88)**				(1.27)
x_7		0.107	0.102	0.065		0.651	0.676	0.879
		(4.46)*	(5.88)*	(355.35)**		(3.47)*	(3.83)+	(3.96)
x_8			0.552	0.609			0.473	0.555
			(1.96)	(372.40)**			(1.20)	(1.59)
x_{10}	1.145	1.071	1.208	1.387	0.046	0.066	0.081	0.077
	(2.62)+	(5.79)*	(8.06)*	(1230.05)**	(1.14)	(3.07)+	(3.45)+	(3.70)
Intercept	123.400	44.820	-0.398	49.040	130.900	1.750	0.180	0.007
R^2	0.795	0.976	0.992	1.000	0.496	0.922	0.955	0.983
F	6.87	29.58	39.72	9156.81	1.31	8.46	6.97	7.25

demand respectively.

Models selected for total gross demand by criteria of important component in explaining dependent variables and t-value test of regression coefficient and the multiple correlation coefficient, are models linear-1, linear-2 and log-linear-2.

$$Y = 27.810 + 0.446x_{10} \quad , \quad R^2 = 0.901 \quad , \quad F = 17.31$$

$$Y = -66.200 + 1.022x_3 + 0.432x_{10} \quad , \quad R^2 = 0.982 \quad , \quad F = 40.63$$

$$Y = 9.58 \times 10^{-7} \cdot x_3^{3.814} \cdot x_{10}^{0.095} \quad , \quad R^2 = 0.863 \quad , \quad F = 4.38$$

Per capita demand models selected by the same criteria are models linear-2 and 4 and log-linear-2.

$$y = 44.820 + 0.107x_7 + 1.071x_{10} \quad , \quad R^2=0.976, \ F=29.58$$

$$y = 49.040-0.116x_2+0.065x_7+0.609x_8+1.387x_{10} \quad , \quad R^2=1.000, \ F=9156.81$$

$$y = 1.750x_7^{0.651} \cdot x_{10}^{0.066} \quad R^2=0.922, \ F=8.46$$

Projection of future damand was made by these models with forecasted parameters of explaining variables. The result of projection is compared with the future demand estimated by past water consumption trend and the estimated value by per capita demand models as shown in Figure 2 and 3. Equation for estimating demand from past consumption was developed as follows;

$$Y = 4095.66 + 4589.09t_i - 22.84t_i^2 + 12.80t_i^3$$

Where t_i: estimation year from the based year 1960.

From the result as shown Figure 2 and 3, it is noticed that the projection values are shown quite different each other. The value of projection is influenced by forecasting parameters of explaining variables as well as the model itself.

CONCLUSION

Total gross and per capita community water demands were projected as shown in Figure 2 and 3. Among the developed models for total gross demand, model linear-1 indicates more adequate value rather than other models which are considered irrelevant for the forecasting purpose. For per capita demand, models linear-2 and log-linear-2 are more significant for the forecasting purpose. In general view, models of per capita demand are considered to be more adequate than those of total gross demand.

The industrial component represented by the number of employees is found to be most important in explaining the both total gross and per capita demand. The industrial water use are analyzed separately, but at the time being sector-wise data, however, are not available on industrial production and water use.

Some possibilities are seen for the extension of the present study to consider each component of the community water demand separately and the forecasting parameters of explaining variables.

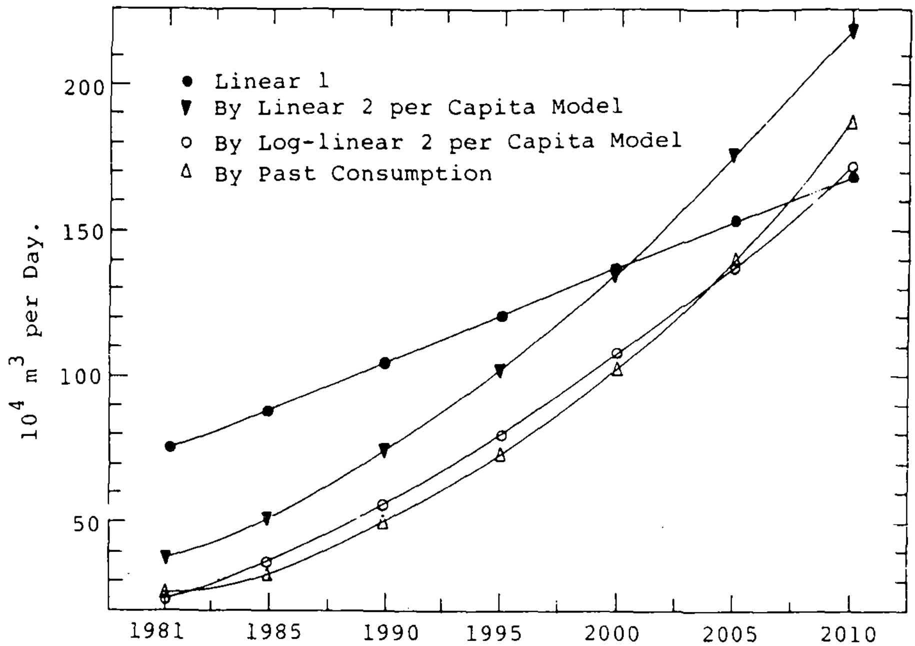

FIGURE 2. TOTAL GROSS COMMUNITY WATER DEMAND.

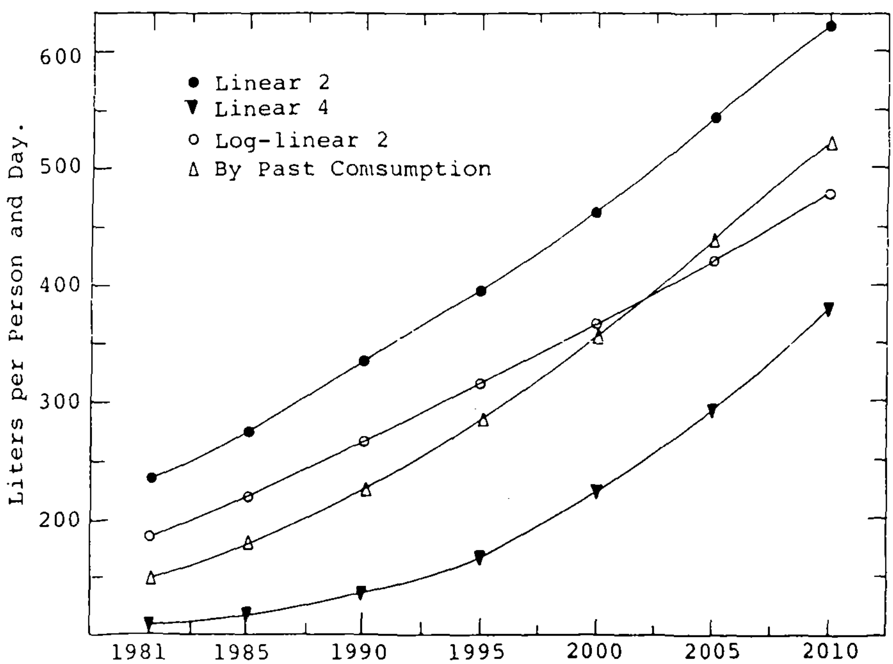

FIGURE 3. TOTAL PER CAPITA COMMUNITY WATER DEMAND.

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PERCEPTION AND ATTITUDE OF THAI VILLAGERS TOWARD AN IMPROVED RURAL WATER SUPPLY MANAGEMENT SYSTEM

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ABSTRACT

The conditions affecting the need for an improved rural water supply management system and the impact which an improved rural water supply system can have on the social and economic well-being of 235 rural households among thirteen villages in Tambon Sara Krue are examined in this study. The need for rural water supply improvement was found to be significantly associated with the villagers' dissatisfaction with the existing water supply, their desire for service, their willingness to contribute to the improvement and their awareness of ill health. No significant association was found between the need for water improvement vs. the number of sources and the awareness of unsanitary condition. The impact which an improved rural water supply management system can have on the social and economic well-being of the village community was found to be associated with medical cost, cleanliness, water conservation, standard of living and community health. However, no significant association was revealed by labor productivity. The feasibility of undertaking an organized rural water supply system was also explored. The analysis indicated that if the project were to succeed, certain social, economic and technical constraints must be overcome.

Keywords: Perception, attitude, improved rural water supply management system, chi-square, association, feasibility, organized rural water supply system, constraints, project and village community.

INTRODUCTION

The provision of potable water to all residents of villages for household consumption on a year-round basis has long been a concern of the Royal Thai Government TAMS (1970), Frankel (1973), NIDA (1978), WRPS-NESDB (1978) and Dworkin and Pillsbury (1980). Since 1966 over 110,000 rural water supply projects, which include shallow and deep wells, storage ponds, reservoirs and tube wells, were completed, and more than twenty-two million inhabitants have benefitted from these installations NIDA (1978). Between 1966 and 1981 more than three billion baht (about $150 million U.S.) had been spent in providing rural water supplies NIDA (1978). Although some 15,000 villages now have organized rural water supply systems, there are at least another 25,000 village communities which have a population of 5,000 or less per village that still do not have access to potable water. According to Chainarong, the Director of the Rural Water Supply Division, Department of Health, only 17 per cent of the rural population in Thailand is served with potable water Chinarong (1978). Despite the continuous investment of millions of bahts in constructing wells to provide portable water for rural areas Unakul and McQuary (1967), Unakul (1971) and WHO (1974), little is known about those communities that have no organized system of rural water supply.

Purpose of study

The purpose of this paper is to investigate one such community by examining the conditions that affect the need for an improved rural water supply management system and by assessing the impact which an improved rural water supply system can have on the social and economic well-being of 236 rural households among thirteen villages in Tambon Sara Krue. In addition, the feasibility of undertaking an organized rural water supply system is also explored.

Data, study area and sample of households

The data for the paper were obtained mainly from the field interviews of a master's thesis that was supervised by the senior author when he was affiliated with the Human Settlements Division at the Asian Institute of Technology, Bangkok, Thailand Azimi (1979).

The area of study was focussed on Amphoe Nong Sua, one of six districts (amphoe) in the province (changwad) of Pathum Thani which is located about fifty kilometers north of Bangkok. The province, which has a population of about 300,000, according to the 1980 Census count, is made up of six districts (amphoe), fifty-seven sub-districts (tambon) and five hundred and five villages (muban).

Amphoe Nong Sua is made up of six sub-districts (tambon). Each sub-district consists of a number of villages called muban. The number of muban in each tambon ranges from eight to thirteen. The tambon that had the largest number of muban was Tambon Sara Krue, which had thirteen muban made up of 985 households. A 24% sample or 236 households was taken from the thirteen villages for interview.

Tambon Sara Krue was selected for study as it had a severe drinking water problem, particularly during the dry season. It was recommended for study by the officials of the Department of Public Health in the province because there was no organized rural water supply program for the area. Most of the villagers here depend on rain as their main source of water supply.

PERCEPTION OF EXISTING RURAL WATER SUPPLY SITUATION IN TAMBON SARA KRUE

Most villagers in Tambon Sara Krue are aware of the existing rural water supply situation. They all know that there is no organized water supply system in their village community. They are aware that their current water supply system is primitive, as virtually all the houses have their sources of water supply contained in earthenware jars located outside the compound. There is no distribution system that delivers water to the individual households. Each household is responsible for its own water supply by fetching water from the nearest surface sources which may be a klong, i.e., an artificial canal, or a pond. Rainwater is collected from the roofs through drain pipes which flow into earthenware water or klong jars. The settlements are located along the klongs, where the distance between the source of water supply and the household is usually not more than 100 meters. Most of the households make four or five trips a day to fetch water. These trips expend time and energy which could be used more productively in other jobs.

Quantity of water supply does not pose a serious problem during the wet monsoon season. It becomes a serious problem only during the dry season which extends from March to May. It is during the dry period that the quantity of supply needs to be supplemented. The shortage arises as a result of insufficient water storage facilities for rain water and for the nearby surface sources. The dryness, due to the lack of rain, leads to dustiness, pollution and poor sanitary conditions in the villages.

Next to the shortage in supply due to the dry season, is the quality of water. Although there is plenty of surface water available in Tambon Sara Krue, the water quality is rather poor. The klong and pond water is very turbid. The water turbidity ranges from 2 to 37 units. According to World Health Organization standards for drinking water, only two villages have suspended matter below 5 units, which is the highest desirable level. Four villages have below 25 units, which is the maximum permissible level, and seven villages have suspended matter that exceeds 25 units.

Water treatment is practiced to some extent in Tambon Sara Krue. Out of 236 households, only 63 treat their drinking water with some chemicals. The rest of the households use either cloth filter or alum or do not treat their water at all.

Although rainwater is used primarily for cooking and drinking, the quality is not entirely satisfactory. The long period of storage, the unprotected water containers, the dust and the pollution from the atmosphere (acid rain) all add to the contamination of the rainwater.

Hand in hand with the inadequate supply of suitable quality of water is the lack of distribution facilities. Many would like to have water piped and delivered to their home. When asked what type of distribution facilities they would like to have for their households to improve the current situation, four types of distribution systems were suggested by the villagers in Tambon Sara Krue. Out of 236 households, 122 (52%) preferred piped distribution (without water meter), 6 wanted tube well (without hand pump), 3 preferred dug-well (with hand pump) and 5 preferred dug-well (without hand pump). The remaining 100 households did not want any improvements to the existing system.

Actually the root of most of the water problems in Tambon Sara Krue is not quantity, quality nor lack of distribution facilities. Rather it is the absence of an organized rural water supply system which has given rise to most of the problems.

AWARENESS OF NEED FOR IMPROVEMENT IN RURAL WATER SUPPLY MANAGEMENT

When questioned whether they thought the existing situation needed improvement or not, 186 (78%) answered yes. They would like to see improvement particularly in the delivery of water to their household. To a large extent their awareness of the need for improvement was influenced by at least six factors, namely, the number of available sources of supply, their dissatisfaction with the existing situation, their desire for service, their willingness to contribute towards the improvement, their awareness of ill health due to contaminated water and the unsanitary condition of the environment. These factors were tested by the use of chi-square to determine the extent to which they were found significant. Four of the factors were found to be significantly associated with the need for improvement at the 0.001 level, while two of them, i.e., number of sources of supply and unsanitary condition of the environment, were found to be not significant. The results of the chi-square tests are shown in Table 1.

Table 1: Results of the chi-square tests

Need for rural water supply improvement vs.	X^2	DF	P
Number of sources of supply	0.089	1	NS
Dissatisfaction with existing supply	11.641	1	0.001
Desire for service	148.019	1	0.001
Willingness to contribute	151.443	1	0.001
Awareness of ill health	12.257	1	0.001
Unsanitary condition	2.266	1	NS

Note: DF stands for degrees of freedom; P stands for level of significance; X^2 stands for chi-square, and NS stands for not significant.

IMPACT OF AN IMPROVED RURAL WATER SUPPLY SYSTEM IN TAMBON SARA KRUE

When the villagers were asked what impact an improved rural water supply system could have on the social and economic well-being of their community, a range of answers was given. These included the impact on the reduction of family medical cost, the improvement in the cleanliness of the environment, the impact on water conservation, the increase in labor productivitiy, the rise in the standard of living, the improvement in community health and the attraction of people to the community. These answers were subsequently tested by the use of the chi-square. The results of the chi-square tests are shown in Table 2.

Table 2: Results of the chi-square tests

Impact of an improved rural water supply system vs.	X^2	DF	P
Medical cost	16.664	1	0.001
Cleanliness of environment	17.345	1	0.001
Water conservation	7.104	2	0.05
Labor productivity	2.761	2	NS
Standard of living	17.563	2	0.001
Community health	17.181	2	0.001
People attraction	16.921	2	0.001

Of the seven factors tested, only labor productivity was found to be not significant. All the six factors, namely, medical cost, cleanliness of environment, water conservation, standard of living, community health and people attraction were found to be significantly associated with the impact of an improved rural water supply system. The significance of these results suggests that considerable benefits could be gained from an improved rural water supply system. The question that needs to be addressed next is what is the feasibility in establishing an organized rural water supply system for Tambon Sara Krue?

FEASIBILITY OF AN ORGANIZED RURAL WATER SUPPLY SYSTEM FOR TAMBON SARA KRUE

It is not easy to develop an organized rural water supply system. In order to undertake such a venture, one needs local input, capital to purchase equipment, trained personnel to operate and maintain the water system, technical skill to guide the villagers and financial resources to maintain a self-supporting system. All these require the involvement of both private and public agencies. Coordination among the different levels of government and communication are imperative for the smooth operation and management of an organized rural water supply system.

Questions may be raised as to what the villagers' desirability is for an organized rural water supply system and how much interest is there for such a system? How willing are they to participate in an organized system? What is their attitude towards forming an association free of government intervention? Can they do an adequate job without government help? What is their preference of public agency participation if the government is to be involved in the program? What kind of cost-sharing arrangements are they willing to undertake? What are the benefits and costs of an organized rural water supply system? What constraints do you think the villagers might encounter in trying to establish an organized rural water supply system?

Desirability of the villagers for an organized rural water supply system

Eighty per cent (189 out of 236) of the villagers expressed their desirability for an organized rural water supply system. Since such a high proportion expressed their desirability, interest in such a system should be considerable. When asked how interested they were in an organized system, 179 out of 236 villagers (76%) said they were interested while 57 (24%) said they were not interested.

Willingness in participation

When questioned how willing they were to participate in an organized system, 173 (73%) said they were willing, 49 (21%) said no and 14 (6%) answered not sure.

Attitude towards forming an association without government intervention

It is interesting to note that the attitude of the villagers towards forming an association without government intervention is quite different from their willingness to participate. One hundred and six (45%) of the households said they were willing in forming such an association, while 60 (25%) said they did not believe in such an association and 70 (30%) said they did not know. Of the 60 who answered 'no' to such an association, 30 gave the reasons for not wanting to form such an association as lack of knowledge, experience and lack of money. Because of the latter reasons, the villagers therefore did not think that they could do an adequate job on their own.

Preference of public agency participation

In Thailand, rural water supply management is very complicated. It comes under the jurisdiction of three ministries: The Ministry of Interior, the Ministry of Industry and the Ministry of Public Health. The latter is concerned with sanitation and the piped water distribution of surface and ground water. The Ministry of Industry is concerned with providing deep tube wells to smaller villages, while the Ministry of Interior is responsible for deep well drilling and ground water exploration. All the three ministries are directly under the National Economic and Social Development Board. Tambon Sara Krue does not come under any of the three ministries because it is too small and too remote.

Any proposed effort to organize a system requires the involvement of the provincial or local government, the village committee, the individual household or a combination of the local government and the village committee. Out of 236 villagers interviewed, 170 expressed preference for the local government to undertake the management of the organized system, 57 showed preference for the village committee, 7 wanted the individual household to manage and 2 preferred a combination of the government and village committee to administer the system.

Cost-sharing arrangements

Cost-sharing arrangements are generally not practiced at the village level as most villages are too poor to work out the arrangement. At the village level, projects are either completely financed by the national government or they are donated by a foreign development agency. The latter is particularly true in the case of water resources projects.

Benefits and costs of an organized rural water supply system

An organized rural water supply system yields benefits as well as incurs costs. For a project to be economically feasible, the benefits must exceed the costs.

Although no formal benefit-cost analysis was undertaken, the benefits that stem from an improved rural water supply system seem to have considerable impact on the social and economic well-being of the villagers in Tambon Sara Krue. This was evident in the assessment of the impact of an improved rural water supply system as shown in Table 2.

Constraints in establishing an organized rural water supply system

While it is socially and economically feasible to undertake an organized rural water supply project, there are also technical and institutional constraints that need to be overcome. Some of these constraints are the lack of technical know-how among the villagers, the lack of skilled personnel, the lack of capital, the lack of willingness for a self-supporting system and the lack of coordination.

Lack of technical know-how

Lack of technical know-how is a common problem in organizing a rural water supply system. In Tambon Sara Krue there is a great need to teach the technical know-how to the villagers. Since most of the villagers are farmers, it is very important that they be trained in the technical skills to install and operate a rural water supply system. Teaching the technical know-how is not an easy task as 75 per cent of the villagers have only an elementary education and 22 per cent of the heads of households have no education at all. Only 2 per cent have a secondary school education, and 1 per cent have up to college or university level. Until the villagers have the education to handle the technical know-how, it is futile to talk about establishing an organized rural water supply system.

Lack of skilled personnel

As we have noted above, most of the villagers in Tambon Sara Krue are farmers with only a bare elementary education. Consequently few have the level of education and experience to deal with the technical and administrative aspects of rural water supply management. In order to overcome this obstacle, it is necessary for the Rural Water Supply Division in the Ministry of Public Health to educate the villagers and provide them with the skills to operate and maintain the system. Providing skilled personnel to administer and manage a rural water supply program is basic for the successful operation and maintenance of any water supply system.

Lack of capital

Although 80 per cent of the villagers expressed a willingness to contribute material and labor for the improvement of rural water supply management, many of them do not really have the financial resources to support what they are willing to contribute. The average household income per year is about 30,000 baht (about $1,500 U.S.). This amounts to approximately 83 baht a day which is less than $5.00 U.S. a day. With such a low income, it is hardly possible for a villager to contribute any money to a project. About 65 per cent of the villagers fall within the 30,000 baht bracket. Unless the villagers have the financial resources available, one should not even try to talk about establishing an organized rural water supply system.

Lack of willingness for a self-supporting system

Even though 80 per cent of the villagers were willing to contribute to the improvement of the rural water supply system, this does not mean that they were willing to contribute to the entire cost of constructing the system. Since the majority of the villagers have such a low income, many were willing to have their taxes raised for the establishment of a self-supporting water supply system. Unless additional income from outside is forthcoming in the form of a gift or loan without attachment, it is extremely difficult for the villagers to raise enough money to construct a self-supporting rural water supply system.

Lack of coordination

In Thailand there are at least six government agencies which are directly involved with rural water supply management. They are: 1) the Department of Local Administration, which provides financial assistance; 2) the Department of Public Works, which looks after provincial deep well drilling projects; 3) the Department of Accelerated Rural Development, which constructs tube wells for smaller communities; 4) the Department of Mineral Resources, which provides deep tube wells to small villages; 5) the Rural Water Supply Division of the Department of Health, which is responsible for the piped water distribution of surface and ground water and 6) the Sanitation Division of the Department of Health, which provides financial assistance for rural institutions, schools, religious places (e.g., temples), health centers and demonstration training in public health. The responsibilities of these agencies entail a wide range of activities which includes financing different types of water supply systems, types of sources and providing training programs to fill skilled personnel demands. Although there are special committees that are set up to help in coordinating the various activities of these agencies, there is still much overlapping and duplication of functions in the execution of activities among the government agencies. Careful coordination of the agency activities is vital to the successful administration and management of any rural undertaking.

CONCLUSIONS AND RECOMMENDATIONS

Since most of the water problems in Tambon Sara Krue are a result of inadequate water supply of a suitable quality during the dry season, lack of storage facilities, absence of distribution facilities and insufficient alternative sources of supply, an obvious guideline for action would be to improve the existing conditions by making more water available to the villagers during the dry season. This would call for improvements in the reduction of the water turbidity, the treatment of water in conformity to WHO standards, the construction of more starage facilities, the establishment of a distribution system and the drilling of more wells to supplement the shortage during the dry period.

The evaluation of the need for improvement in the existing situation and the assessment of the impact which an improved rural water supply system can have on the social and economic well-being of the villagers in Tambon Sara Krue suggest that an organized rural water supply system could be established. A feasibility analysis of the latter indicated that if the undertaking were to succeed, certain technical and institutional constraints must be overcome. These include the technical know-how, the lack of skilled personnel, the lack of capital, the lack of willingness for a self-supporting system and the lack of coordination.

Although a majority of the villagers expressed preferences for the government in undertaking the initiative to establish an organized rural water supply system, it is suggested that the administration and management of the system be a joint effort of both the Rural Water Supply Division of the Ministry of Public Health and that of the Village Committee rather than it be left entirely to the responsibility of the villagers and their committee.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

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FLASH FLOOD FORECASTING IN THE CEVENNES REGION IN FRANCE - A CASE STUDY

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ABSTRACT

The Cevennes region in Southern France was selected as a pilot area for the development of flash flood forecasting systems because of the heavy rainfalls and rapid concentration of the flows which occur in this region of steep upper basins and flat lower basins. The flood forecasting system consists of a real-time data acquisition subsystem, a hydrologic forecasting subsystem and an alert information dissemination subsystem. The first two subsystems are discussed in this paper and the emphasis is on the formulation of the second subsystem. The proposed hydrologic forecasting subsystem consists of tow models in series. The first is a nonlinear model which transforms the total rainfall into an effective rainfall. This, in turn, serves as an input to the second model which is a linear transfer function model, the coefficients of which can be estimated recursively in real time. The recommended models are selected on the basis of the parsimony of parameters, the standard errors of forecasting at the identification stage and at the verification stage and on the whiteness of the residuals. A wide variety of models were tested using data from the basin of the Gardon D' Anduze.

Keywords: Flash floods, flood forecasting, real-time forecasting, rainfall-runoff relationship, recursive estimation.

INTRODUCTION

Flooding is the most important natural disaster in metropolitan France. The annual economic loss is of the order of 2 to 5 X 10^9 French Francs. Some floods may be due to slow but protracted rains over large areas while others may be due to intensive storms over small basins producing flash floods. This study is concerned with the latter and in particular with flash floods in the region known as the Cevennes in southern France.

This region is on the south-eastern slope of the Central Mountains (Massif Central) and mostly north of the historically famous Roman acqueduct of the Pont du Gard, near the City of Nimes (Fig. 1). The Gardon D' Anduze, with a tributary area of 545 Km^2, was used as a test basin in this study.

Climatologically the Cevennes mountains are under Mediterranean influence. Meteorological perturbations are fed by warm moist air from the Mediterranean sea. Large rainfall volumes and intensities are common, particularly in the autumn. Precipitations of 400 mm in 2 to 3 days are not uncommon and rainfall of 800 mm in 24 hours has been observed. A point rainfall of 1 m in a day is a definite possibility. A point rainfall intensity of 100 mm/hr was observed in 1976, and the corresponding average rainfall of the Gordon d' Anduze was 32 mm/hr.

The steep relief of the Cevennes mountains produces a rapid concentration of the flows resulting in flash floods. With an average flood wave celerity of the order of 8 Km/h, the response time of these basins is very short, typically of the order of 1 to 6 hours, depending upon the size of the basins. As an example, the flow of the Vidourle River at Sabatier with a tributary area of 200 Km^2 passed from $30m^3/s$ to $500m^3/s$ in one and a half hours in spite of a reservoir at Ceyrac which controls $45km^2$ of the basin.

The time available for alerting the endangered population is thus extremely short, a few hours at the most. For this reason the Gard department was chosen in the late seventies as a test area to establish a pilot flood forecasting system. This system supplements a program of construction of flood control dams and levees. Five dams have been built since 1967 and four more are under study.

The flood forecasting system consists of three subsystems:

1. the real-time data acquisition subsystem

2. the hydrologic forecasting subsystem

3. the alert information dissemination subsystem

The hydrologic forecasting subsystem is the main theme of this paper.

DATA ACQUISITION SUBSYSTEM

The data acquisition network consists of 17 stations for the real-time measurement of rainfall depths, stream stages, temperatures, etc., which are radio-transmitted with the help of two relays to the central forecasting station at Nimes (Fig. 1). In addition, there is a large number of rainfall and streamgaging stations which are not radio transmitted, but which have been used for special studies. For example Tourasse (1981) in a study of the dynamics of autumn precipitations in the Cevennes region used 59 stations, while Lebel (1984) in a study of the spatial distribution of the rainfall extended the area of interest to the West and the South and used data from 97 recording rain gages.

The data acquisition subsystem consists of 3 parts

1. a clock which starts the data acquisition cycles and the programs which test for the data coherency and completeness, and detect if a precipitation or a river stage exceeds a predetermined threshold.

2. a data management system which will record the information in more detail when a threshold has been exceeded.

3. a set of programs which gives a tabular and graphical presentation on a screen and on paper of the hydrometeorological state of a flood event. These include maps of the 1 hour- and 24 hour-rainfall over selected basins, hyetographs and stage hydrographs at desired stations, historical rainfall and stage information. An instantaneous "snapshot" of the hydrometeorological state of the basins is thus available at any time at the forecasting center.

HYDROLOGIC FORECASTING SUBSYSTEM

The purpose of the hydrologic forecasting subsystem is to estimate the flows or river stages at several critical points of the basin(s) one and several time steps ahead. Two approaches are possible. One is deterministic and requires the detailed modeling of the physical process involved in the transformation of rainfall to runoff for small homogeneous areas and the routing of the flows from these areas to the basin outlet. This approach would require a much more extensive data base than is available, and it is not, in general, practical for real-time forecasting. The second approach which treats each basin or subbasin globally as a system was used in this study. It was assumed that the system consists of two components in series. The first component, which is nonlinear, determines the production function, namely, the part of the precipitation or effective rainfall which will become runoff. The second component transforms this effective rainfall into runoff by means of a linear transfer function.

Production Function

The purpose of the production function is to transform the total rainfall that occurred during a time interval $P(t)$ into an effective rainfall or rainfall excess $PE(t)$ which will reach the basin outlet as surface runoff. Let $W(t)$ be the "losses" or the difference between $P(t)$ and $PE(t)$, thus

$$PE(t) = P(t) - W(t). \qquad (1)$$

$W(t)$ represents the part of the rainfall that is stored locally as interception, depression storage, storage in the upper layers of soil. This storage will eventually be depleted by deep percolation and by evapotranspiration.

The theory of infiltration and evapotranspiration is, at present, too data intensive for the purpose at hand. It is therefore necessary to resort to an empirical formulation or to a conceptual modeling. The latter approach was selected. The surface storage, $S(t)$, is represented by a linear reservoir with an inflow $W(t)$, and an outflow equal to the deep percolation $I(t)$

$$S(t) = S(t-1) + W(t) - I(t)$$

with $\quad I(t) = \gamma[S(t-1) + W(t)]$

thus $\quad S(t) = (1-\gamma)[S(t-1) + W(t)]$

The storage available or moisture deficit is

$$D(t) = S_{max} - S(t-1) = \gamma S_{max} + (1-\gamma)S_{max} - S_{t-1}$$
$$= \gamma S_{max} + (1-\gamma)\ [D(t-1) - W(t-1)]$$

Let $S_{max} = D_1$, the initial moisture deficit, and $\alpha = 1-\gamma$, then

$$D(t) = \alpha\ [D(t-1) + W(t-1)] + (1-\alpha)\ D_0 \qquad (2)$$

The loss $W(t)$ is expressed as a function of $D(t)$ and $P(t)$ such that $PE(t)$ tends to $P(t) - D(t)$ for large rainfalls $P(t)$; the runoff coefficient $[PE(t)/P(t)]$ increases with the rainfall intensity, i.e. with $P(t)$ and the runoff coefficient decreases when $D(t)$ increases. The following relationship satisfies these requirements:

$$W(t) = D(t)[1-\exp\ (-\beta \frac{P(t)}{D(t)})] \quad , \quad 0 < \beta < 1 \qquad (3)$$

The rainfall excess is thus estimated by means of equations 1, 2 and 3 which have 3 parameters α, β and D_0. In a study of the same geographical region, Versiani (1983) estimated the rainfall excess by means of an iterative multiple regression and deconvolution method proposed by Guillot and Duband (1980). From this, the optimized value of β was found to be 0.89 and D_0 was related to the accumulated three day antecedent precipitation SP (in 10th of mm) by

$$\ln D_0 = C + a.SP$$

with a = -0.00229566, c = 7.00913 and regression coefficient of -0.848. The best value of α, found by trial and error, was 0.95.

Transfer Function

The dynamics of lumped linear hydrologic systems may be examined, for example, by the differential equations that connect the input (effective rainfall) and the output (runoff), as proposed by Chow and Kulandaisway (1971, 1978), or alternatively by means of the state space formulation and the Kalman filter (Szollosi-Nagi, 1976), or by means of linear reservoirs (Bastin, 1984). Making use of the first approach, the output $y(t)$ is related to the input $u(t)$ by the symbolic equation

$$y(t) = \frac{M(D)}{N(D)}\ u(t) \qquad (4)$$

where $N(D) = \alpha_n D^{n+1} + \alpha_{n-1} D^n + \ldots \alpha_0 D+1$ and $M(D) = \beta_m D^{m+1} + \beta_{m-1} D^m + \ldots + \beta_0 D-1$ in which $D^m = d^m/dt^m$.

A finite difference equation approximation to the differential equation 4 is

$$y(t) = \frac{B(z^{-1})}{A(z^{-1})}\ u(t-d) + v(t) \qquad (5)$$

where $A(z^{-1}) = 1 + a_1 z^{-1} + \ldots + a_p z^{-p}$ and $B(z^{-1}) = b_1 z^{-1} + b_2 z^{-2} + \ldots + b_q z^{-q}$, z^{-1} is the backward shift operator such that $z^{-1} y(t) = y(t-1)$, d is the time delay between input and output and v(t) is the error.

The error terms may be represented by an autoregressive - moving average (ARMA) Model (Box and Jenkins, 1976):

$$D(z^{-1})\ v(t) = C(z^{-1})\ e(t) \tag{6}$$

where e(t) is a white noise, $C(z^{-1}) = 1 + c_1 z^{-1} + \ldots c_{qe} z^{-qe}$ and $D(z^{-1}) = 1 + d_1 z^{-1} + \ldots + d_{n_d} z^{-n_d}$. The general model can thus be represented by

$$y(t) = \frac{B(z^{-1})}{A(z^{-1})}\ u(t-d) + \frac{C(z^{-1})}{D(z^{-1})}\ e(t) \tag{7}$$

ALGORITHMS UTILIZED

Two types of algorithms were used. The first was a Recursive Least Squares (RLS) algorithm to estimate the parameters of the model

$$A(z^{-1})\ y(t) = B(z^{-1})\ u(t) + v(t) \tag{8}$$

The computer program was written so that 5 inputs could be used:

$$A(z^{-1})\ y(t) = B_1(z^{-1})\ u_1(t) + \ldots + B_5(z^{-1})\ u_5(t) + v(t). \tag{9}$$

In an optional second application of the RLS algorithm, using v(t) instead of y(t) as input and taking $B(z^{-1}) = 0$, the error v(t) can be modeled as an autoregressive (AR) process

$$D(z^{-1})\ v(t) = e(t)$$

This yields the Generalized Least Squares (GLS) model

$$A(z^{-1})\ y(t) = B(z^{-1})\ u(t) + \frac{1}{D(z^{-1})}\ e(t) \tag{10}$$

In the second algorithm the error v(t) is modeled by a moving average process. A Recursive Maximum Likelyhood (RML) algorithm was used to estimate the parameters of the model

$$A(z^{-1})\ y(t) = B(z^{-1})\ u(t) + C(z^{-1})\ e(t) \tag{11}$$

This is known as the ARMAX model, autoregressive (AR) model in y, the error being represented by a moving average (MA) process, and with the exogenous (X) input. The computer program was written so that 5 inputs could be used:

$$A(z^{-1})\ y(t) = B_1(z^{-1})\ u_1(t) + \ldots + B_5(z^{-1})\ u_5(t) + C(z^{-1})\ e(t) \tag{12}$$

With the computer programs used, the AR error model of the GLS model required two consecutive applications of the RLS program while with the MA error model, all the coefficients were determined in a single application of the RML program. A detailed discussion of the algorithms used and of the usage of the computer programs can be found in Delleur (1984).

APPLICATION TO THE GARDON D'ANDUZE

Two types of inputs were used. In the first type a single input was used, namely the average hourly rainfall over the Gardon D'Anduze. These average rainfall values were obtained by a spline interpolation (Lebel, 1984). Ten flood episodes that occurred between 1972 and 1976 were used for the parameter estimation of the models, and seven flood events that occurred between 1976 and 1979 were used for the verification of the model. In the second type of application the hourly point rainfalls and the hourly flows in an upstream subbasin were used as inputs. In all cases the output was the hourly flow of the Gardon at Anduze.

Mean Total Rainfall Input

An exhaustive study was performed of all possible models of the types of equations (10) and (11) with 1 to 6 parameters with and without error models. The input was the average total rainfall without losses. Table 1 lists the standard errors at the identification stage σ_i and at the verification stage σ_v. The best model at the identification stage is not necessarily the best model at the verification stage.

Table 1. Standard errors as a function of the number of parameters.

No. of Parameters	Best Models σ_i m^3/sec	σ_v
1	30	30
2	22	25
3	20	23
4	19	23
5	19	24
6	19	23

The optional number of parameters is seen to be 4 as no additional improvement in performance is obtained by a further increase in the number of parameters. As a matter of interest, an 18 parameter unit hydrograph model yielded a much lower performance than the above four-parameter models as measured by their standard errors $\sigma_i = 65m^3/sec$ and $\sigma_v = 105m^3/sec$.

The best four-parameter model without error model is

$$Q(t) = 1.61\ Q(t-1) - 0.97\ Q(t-2) + 0.28\ Q(t-3) + 0.31\ P(t-2) + e(t) \qquad (M1)$$

with $\sigma_i = 19m^3/sec$ and $\sigma_v = 24m^3/sec$. The best four-parameter model including an error model at the identification stage is

$$Q(t) = 0.86\ Q(t-1) + 0.20\ P(t-2) + 0.37\ P(t-3) + 0.70\ e(t-1) + e(t) \qquad (M2)$$

while at the verification stage the best performing model is

$$Q(t) = 0.87\ Q(t-1) + 0.10\ P(t-2) + 0.47\ P(t-3) + v(t) \qquad (M3)$$

$$v(t) = 0.51\ v(t-1) + e(t)$$

Mean Effective Rainfall Input

The effective rainfalls were calculated using the production function equations (1), (2) and (3) with $\alpha = 0.95$, $\beta = 0.89$ and D_0 based on the antecedent 3-day mean precipitation. Table 2 shows the importance of the production function as it decreases the standard error at the estimation stage and at the verification stage by about 18%. The models M4, M5, and M6 of Table 2 do not include any error submodel. Figure 2 shows, at the verification stage, the one-step forecast obtained with model M4 (dotted line) and the observed flow for flood event 7650 which occurred in 1976.

Table 2 - Standard errors obtained with total rainfall and with rainfall excess.

	Model	Total Rainfall σ_i	Total Rainfall σ_v	Effective Rainfall σ_i	Effective Rainfall σ_v
M4	$Q(t) = a_1Q(t-1)+a_2Q(t-2)+a_3Q(t-3)+b_1PE(t-2)$	19.5	23.7	18.7	22.6
M5	$Q(t) = a_1Q(t-1)+a_2Q(t-2)+b_2PE(t-2)+b_3PE(t-3)$	20.1	24.6	19.8	23.8
M6	$Q(t) = a_1Q(t-1)+b_2PE(t-2)+b_3PE(T-3)+b_4PE(t-4)$	23.9	26.8	22.4	25.4

As shown in Table 3, the effect of adding a one-parameter error submodel is small for the better models, that is for those with a small standard error. In the case of model ML the decrease in standard error was only at the identification stage. The addition of an AR model was found to be more effective than an MA model at the verification stage. An explanation is that the error has a basic AR structure, but at the identification stage the RML program estimates all the parameters together including the MA parameter, whereas two consecutive applications of the RLS program were used to estimate the basic model parameters and the AR error parameter, thus giving a slight advantage to the MA model at the identification stage.

Table 3 - Effect of the error model.

Basic Model	σ_i m^3/sec	σ_v	Error Model	Combined Model σ_i	Combined Model σ_v
M4	18.7	22.6	AR	18.5	25.8
			MA	18.5	27.2
M5	19.8	23.8	AR	19.2	23.1
			MA	18.1	47.7

In general, the principal effect of the error model is to make the residual error closer to a white noise as can be seen through the use of the correlogram of the residuals. The whiteness of the residuals becomes more important for the multi time step forecasting.

Forecasting Several Time Steps Ahead

Forecasting 2, 3 on more time steps in advance can be performed by concatenation of 2, 3 or more one step models or by using a program that permits the direct estimation of the parameters in a multitime step model. The

RML program had that capability. As an example, two concatenations of model M4 with an added AR noise model gave a verification standard error of $66.3 m^3/sec$. In contradistinction, the model

$$Q(t+2) = 1.06\ Q(t) - 0.34\ Q(t-1) + 1.44\ PE(t) + 2.61\ PE(t-1) + e(t+2)$$

with the parameter estimated in one step by the RML program gave a value of $\sigma_v = 52.6 m^3/sec$ or an improvement of $3.7 m^3/sec$.

Multiple Input Models

The Gardon D'Anduze has three main tributaries, the Gardon de St. Jean., the Gardon de Miallet and the Salindringue River. A station at Soucy measures the flow of the upstream half of the St. Jean subbasin. The flows at this station and the rainfalls at four stations distributed over the remainder of the basin of the Gardon D'Anduze were selected as inputs. The best model contains 2 autoregressive terms for the outflows at Anduze, Q, 2 terms involving the flows at Soucy, Q_1, and the observed precipitation at the four stations, which are inputs 2 to 5 labeled as $P_2,..,P_5$, and an autoregressive noise submodel.

$$Q(t) = 1.34\ Q(t-1) - 0.45\ Q(t-2) - 0.06\ Q_1(t-1) - 0.17\ Q_1(t-2)$$

$$+ 0.10\ P_2(t-2) + 0.09\ P_3(t-2) + 0.08\ P_4(T-2) - 0.03\ P_5(t-2) + v(t)$$

$$v(t) = 0.02\ v(t) + e(t).$$

The standard errors are $\sigma_1 = 24.2$ and $\sigma_v = 24.3 m^3/sec$ which is of the same order of magnitude as the error obtained with model M5. This shows that with appropriately chosen rain stations and/or flows in upstream tributaries, adequate forecasts can be made, bypassing the calculations of the average rainfall over the basin, and the estimation of the rainfall excess.

CONCLUSION

A family of models was studied for real - time forecasting of flood flows one and several time steps ahead. The models consisted of a non-linear conceptual model for the transformation of the observed rainfall into an effective rainfall, followed by a linear input-output model whose parameters are determined recussively either by a least squares algorithm or by a maximum likelihood algorithm. In the application to the Gardon D'Anduze, the optimum number of parameters of the transfer function was found to be four. The models consisted of a low order autoregression on the outflows, one or two weighed values of the delayed rainfall excess and a first order autoregressive error submodel. The best results were obtained using the spline average rainfall from which the effective rainfall was estimated by the conceptual model. Results almost as good could also be obtained using appropriately selected point rainfalls and outflows from upstream subbasins. A number of models of this type must be estimated and verified with inputs of different types and at different stations to cover the eventuality that one or more stations might be disabled in an extreme storm event. It is expected that models of this type will be implemented in the very near future in the Cevennes region.

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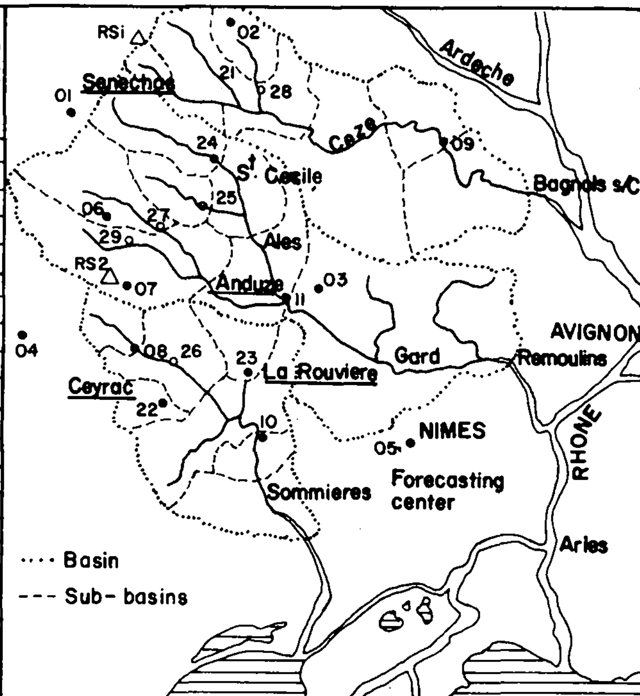
BASIN	Station Nº	Power	Reservoir	Stage	Rain	Wind	Temperat.	Radiation	Psychrom.	Pressure
CEZE	01	S			•	•	•			
	02	S			•					
	09	S		•	•					
	21	R	•	•	•					
	28	-	o	o	o					
VIDOURLE	08	S		•	•					
	10	R		•	•		•			
	22	S	•	•	•	•	•			
	23	S	•	•	•					
	25	-	o							
	04	S			•	•	•			
	05	R			•	•	•	•	•	•
GARD	03	R			•	•	•	•	•	
	06	S			•	•	•	•	•	
	07	S			•					
	11	S		•	•		•			
	24	S	•	•	•		•			
	25	S	o	o						
	27	S	o	o						
	29	S	o	o						

S. Solar R. Network



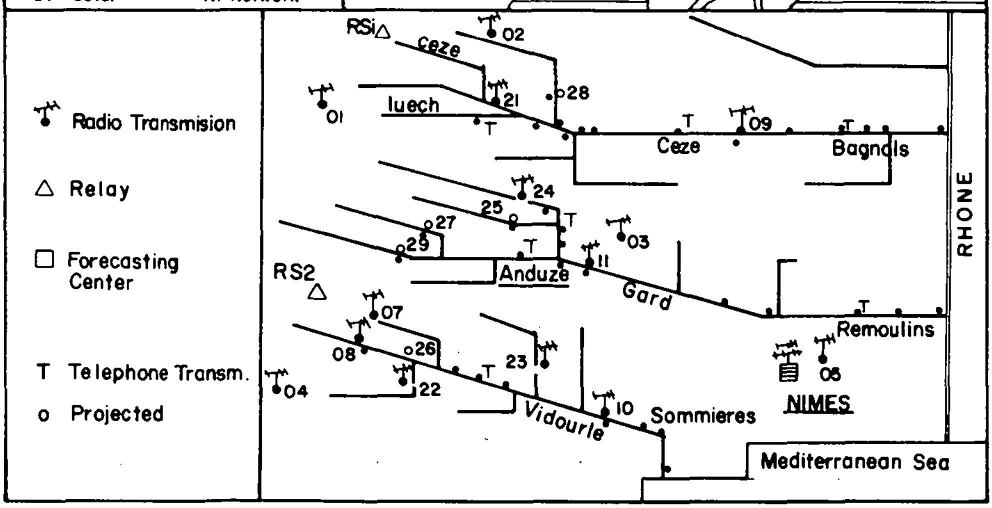


FIGURE I. Rain and stream gages for flood forecasting in the Cevennes region

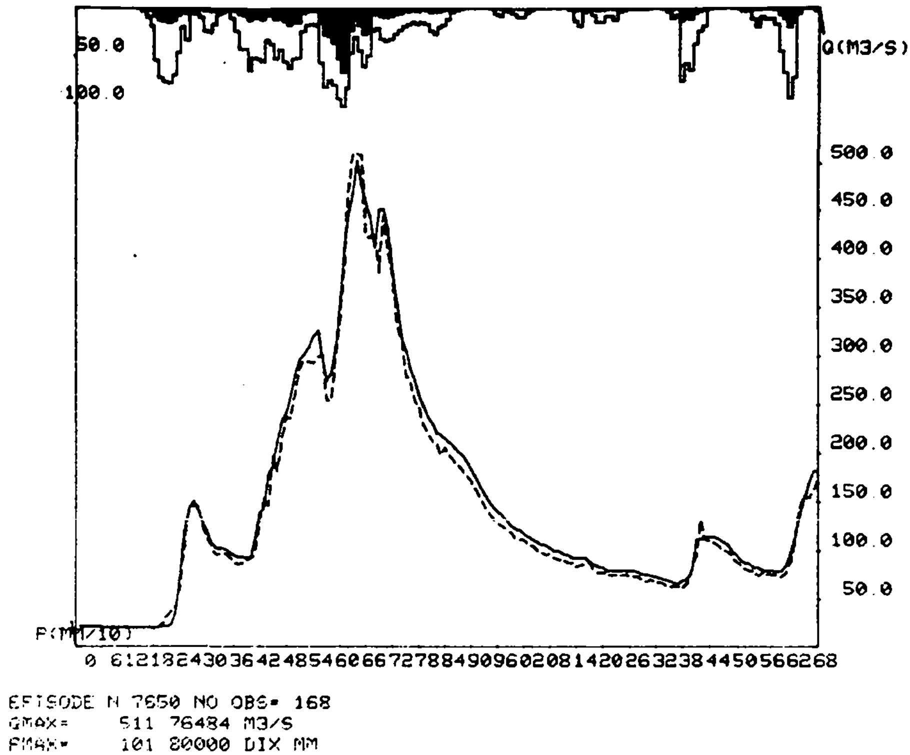


Figure 2. One step ahead forecast (dotted line) by model M4, observed hydrograph, observed and effective hyetographs for event 7650. $Q_{max} = 511.76 m^3/sec$ and $P_{max} = 101.8 \times 10^{-1} mm$, duration 168 hours.

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THE IMPACT OF WATER RESOURCES PUBLIC POLICIES ON INTERNATIONAL RIVER BASIN CONFLICTS (*)

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ABSTRACT

The satisfaction of national water needs frequently requires the resolution of international water use conflicts. Success or failure of negotiations is conditioned not only on the availability of the resource, but the degree to which the public policies of each of the countries involved facilitate, constrain, or retard settlement. An analytical model of the policy process that governs the resolution and/or management of an international river basin conflict, applicable to countries at any level of development, is presented. The model identifies the components of a cycle that begins with the identification of needs requiring official government action, and ends with the production of policy outputs to allocate resources for the satisfaction of the identified needs. The model is used to analyze the conflict between Mexico and the United States over the salinity of the Colorado river, as an illustration of its usefulness for applying it to other conflicts.

INTRODUCTION

This paper analyzes an essential yet neglected aspect of international river basin conflicts: the effect that the political decision-making process of each of the countries involved has on the outcome of negotiations. Description of a policy analysis model will be presented, applying it to the particular case of negotiations between the U.S. and Mexico for control of increasing salinity in the Colorado River. The model examines the role of the political institutions guiding the process and the cause/effect relationship of actions taken.

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* The authors are solely responsible for opinions expressed in this paper.

There is substantial literature describing the legal and institutional aspects of river basin conflict resolution (Teclaff, 1972, UNDSA, 1975). These works occasionally provide some analysis of the intra- and international policy processes. Conceptual frameworks, matched with case studies of international conflicts have sharpened the understanding of the factors facilitating settlement (Le Marquand, 1977). A research design is lacking, however, that will accommodate the governmental processes of any country at any level of development, regardless of its background in law or custom, and to explain the impact of governmental decisions on international water conflicts.

GENERAL CHARACTERISTICS OF THE MODEL

The analytical model used in this paper derives from Easton (figure 1; Easton, 1965). It emphasizes the role of the bureau (administrative unit) as the producer of outputs, policy or program, affecting the needs in the "environment" of a political subsytem (figure 2). The policy process generally consists of an array of overlapping subsystems. Issues activating each subsystem may be broad, involving a number of bureaus, or they may be narrow involving a small organization. The policy subsystem model used in this analysis refers to decisions made by a national government, but it can adapt easily to the analysis of decisions made at the state or local level.

There are some variables of the same kind operating with respect to each of the model's components. For example: the bureau always produces one or more of four major types of outputs: (1) provides funding or the means to plan, develop, operate and/or manage programs, projects, facilities, etc.; (2) regulates public and/or private activities; (3) conducts or supports research and data-gathering; and (4) provides technical assistance to improve public or private activities. Responding to environmental needs and to goals set by various expert agents or groups, demands are articulated by those influencing governmental decisions (the "gatekeepers", in Easton's terms) after which decisions are made. These decisions must be made by the governments in accordance with constitutional or institutional authorizations to executive, legislative and judicial branches. Influencing factors may be political parties, interest groups or other entities motivated by private profit or their own definition of public benefit.

In theory, the policy sybsystem reacts to any environmental need be it small or severe. In practice the process responds at different rates subject to a host of variables.

The use of the policy subsystem model to analyze the U.S.-Mexican conflict will be discussed after examining each country's policies on water resources and the events leading to the resolution of the salinity control issue, with the adoption of Minute (Resolution) 242, "Permanent and Definitive Solution to the International Problem of the Salinity of the Colorado River," on August 30, 1973.

Water Resources Policies - Mexico and the United States

The disparate water resources policies in effect in each country have often been broadly described. Analyses of the political decisionmaking processes in each country have been less frequent, tending to deal more with the legal and institutional processes of the United States, rather than those of Mexico (Hundley, 1966; Mann, 1975). Generally speaking, the United States system is described as "decentralized" and "fragmented" in contrast to the "centralized," "authoritarian" regime in Mexico. The purpose of the present analysis is to examine the decisions made to solve a specific conflict, and the interplay of all participants in the process including river basin planners and those who assumed or were given responsibility for government decisions. Quantification techniques, possible to use with some aspects of policy systems analysis, are not attempted in this paper. The approach used cites trends and patterns, useful in understanding current and future outcomes of this particular kind of political process.

Prerogatives in U.S. water resources policies are shared among local, state and national governments and most of the water resources issues are resolved by the states. Legal and political institutions are somewhat different in each state, although with overall regional similarities. However, a national policy has indeed developed and evolved, including some major aspects of water resources use and management. Substantial federal support has been given to flood control, land reclamation, water pollution control, and research, among other national and regional needs. Overall, the policy outputs identified in figure 3 covering all of the four major categories of programs have a long and continuing history of national support, although with some changes in time, as they have been produced by a number of political subsystems. This continuity in policy is evident in the efforts made through the years to resolve the salinity conflict between the U.S. and Mexico.

In Mexico, water resources policies, like other major policy issues, are controlled at the federal level. Historical and cultural factors contributed to this, as shown by Article 27 of the 1917 Constitution, which sanctions public control over most natural resources. The National Water Act of 1934 specifies priority use for water resources and constitutes a national plan for their management. The 1947 Health Engineering Act gave the central government exclusive power to plan and operate a national water delivery system. A 1957 law established a permit system for ground-water use (Mumme, 1982). The government of Mexico has a strong national government that personalizes the rule of its President during his single six-year term. The strength of the executive branch is reflected in the allocation of water issues to one of the major bureaus, the Secretaria de Recursos Hidraulicos, established in 1947, with supervision and general authority over the administration of water development, rights and related land decisions. The Federal Water Law of 1971 further consolidated federal water management under the Secretaria as part of a general reform, as did its later merger with the Ministry of Agriculture.

Both the United States and Mexico have pursued "good neighbor" policies in their dealings with each other, preferring agreement to confrontation. The leadership in both countries, at times, has had to ameliorate internal conflicts before resolving their international difficulties.

RIVER BASIN CONFLICTS BETWEEN THE U.S. AND MEXICO

The lengthy negotiations that produced the initial water boundary settlement between the U.S. and Mexico and the establishment of the International Boundary and Water Commission (IBWC) in 1944, required as much diplomacy on the part of the U.S. Department of State in dealing with

Colorado River Basin States as in negotiating across the border. Many interstate problems had to be dealt with before a negotiated settlement with Mexico could be accepted by the Congress. On the Mexican side, the expectation was that once a settlement for the control of the problems in upstream of the Rio Grande been reached, it would be easier to resolve the downstream problems on the Colorado River. Time would prove that the process was far more complicated than expected. What follows is a description of the "bureau" (IBWC) as it relates to the policy process of resolving the problems of the Colorado River followed by a brief review of the negotiations betweeen the U.S. and Mexico.

IBWC - Structure and Resources

The bureaus or agencies engaged in international settlement of differences between countries and at the "production end" may be foreign offices or any other bureaus with relevant functions. In the case of the United States and Mexico, an international agency, the IBWC, was established in 1944 and it has been the instrument for settlement of disputes over the three river basins they share -- Rio Grande, Colorado, and Tijuana. Each section of the Commission, one operated by the United States and the other by Mexico, functions as a national bureau and is under the jurisdiction of the respective foreign offices. It is staffed primarily by technical and legal advisers with an "Engineer Commissioner" heading each section. Both country's Commissioners and top staff have been career appointees enjoying longevity. The Sections function as one with respect to certain quasi-adjudicatory and administrative matters. Decisions must be ratified by the respective governments before they can take effect. Each Section conducts it own investigations and monitoring activities. They construct (or fund the construction of) major water works and sanitation facilities on their respective sides of the shared river basins (Furnish, 1975).

The Salinity Crisis - U.S. and Mexico Policymaking

The first problem of major proportions confronting the IBWC was the increasing salinity of the Colorado waters received by Mexico which began in 1961. The sudden rise resulted from pumping out briny water from waterlogged irrigated cropland at the Wellton Mohawk Irrigation Project in Arizona and the construction of the Glen Canyon Dam. Excessive crop losses were sustained in the Mexicali Valley as recordings of salt content rose from 800 parts per million (ppm) in 1960 to 1500 ppm two years later. Seven per cent of all irrigated crops in the country were cultivated in the Valley and heavy damages were reported during the early 1960s (Gonzalez-Leon, 1975; Sepulveda, 1972; Brownell, 1975).

Through interim agreements, the acceptable load of salt was set during the period of negotiations at 1250 ppm. The final settlement provided for Mexico to receive waters with a salinity level of 1150 ppm, plus or minus 30 ppm, or 100 ppm higher than that at the Imperial Dam on the United States side (IBWC, 1973).

Initially, the U.S. State Department refused to acknowledge responsibility for the increased salinity of the Colorado, or that its occurrence was a violation of the 1944 Treaty. But when the matter was brought to the attention of President John F. Kennedy in 1961, a decision to settle the conflict was soon reached. The IBWC assembled a panel of experts from both countries to make recommendations for the resolution of the problem. A series of temporary solutions over the next 12 years followed, with U.S. and Mexican Presidents assuming strong leadership roles in the negotiations to bring about a permanent settlement of the conflict.

In 1972 the Brownell Task Force was established and with the revived "Committee of Fourteen" (two members per basin state) an agreement was reached in 1973 recommending the construction of a desalting plant and other measures. Ratification and implementation of Minute 242 describing the agreement hinged not only on the U.S. State Department interpretation of international law, but also on the willingness of the Colorado River basin states to exercise their influence on Congress for implementing the proposed solution.

Two bills were developed. One, sponsored by the Administration, gave the IBWC (and therefore the State Department) the authority to construct the plant. The other bill, introduced and supported by basin state Representatives in Congress, authorized the Bureau of Reclamation to construct the desalting plant, with costs to be borne exclusively by the Federal Government. Meanwhile, the newly created Environmental Protection Agency identified salinity as a pollutant under the Federal Water Pollution Control Act Amendments of 1972. Its enforcement provision meant that action would have to be taken to control salinity within the United States, beyond that proposed at Yuma. The bill supported by the basin states was passed by the Congress and signed into law by President Nixon on June 26, 1974. Currently, construction of new Federal projects is in a holding pattern with the Yuma Desalting Plant itself not expected to be in operation until the end of the decade. Figure 3 provides a summary of the negotiations between the U.S. and Mexico using the policy subsystem as the frame of reference.

The IBWC is still operating, and its activities have been examined by many authors. A close "clientele" relationship has been maintained by the career-staffed Section with Congressmen from the basin states and from border areas in Texas that have profited from IBWC activities. This has fostered a close relationship between Federal bureau-water development reclamation in the western states with the agricultural and other economic interest groups as well as with local and state officials. It should be noted that the Commission is substantially independent of the State Department.

The Mexican Section has been found to be less "clientele" oriented and more closely supervised by its foreign ministry. The task of developing constituent support for Mexican proposals fell entirely on the Foreign Ministry although national concerns about the effect of salinity on farming were conveyed to that Ministry by the Agricultural Minister.

ANALYSIS OF THE U.S.-MEXICO POLICY SUBSYSTEMS

The policy process governing the negotiations between the U.S. and Mexico over the salinity problems of the Colorado River is depicted in the flow diagram of figure 4, where the main actors are presented in simplified form. In the U.S. subsystem the need for government action was brought about initially by Mexico's complaints. Later domestic salinity problems became important as well. The goal setters recognized the Mexican claims initially; U.S.

national needs developed later. Inputs from Federal and state planners, administrators, and scientists as well as IBWC's experts were important in defining goals agreeable to both countries. The main influencing agents were, and still are, the basin states private and public interests, with the "good neighbor" policy guiding the State Department's actions. The executive branch of the U.S. government was a key factor in the decisionmaking part of the process, but Congress had the final word, especially in adjudicating authority and funds.

On the Mexican side, damage to agriculture was the triggering factor, with goals being mainly set by federal agencies, with limited input from scientists and administrators. The main influencing agents were the Secretaria de Recursos Hidraulicos, and farmer and business groups of the ruling party Partido Revolucionario Institucional. The decisionmaking was centralized in the Foreign Ministry under the direct oversight of the President. The binational bureau, IBWC, was given limited authority to implement policy in both sections. The U.S. section is strongly influenced by basin state interests (legislative branch); meantime the bulk of implementation responsibilities falls on Federal agencies (executive branch). On the other hand, the Mexican section is responsive only to the executive branch through the presidency and two federal agencies, with very limited direct local input.

CONCLUSIONS

This model has been found to be helpful in understanding and assessing: (1) the main influential factors that dominate the outputs (actions taken) of the political system; (2) the relationships between the actors that control the decisionmaking process; and (3) the main inputs used by influencing agents and decisionmakers. It is further postulated that, in dealing with international conflicts, a preliminary systematic examination of the policy process using the framework provided by the model for each country involved in the strife, gives a better perspective on how negotiations can be facilitated, delays avoided, and outcomes bettered.

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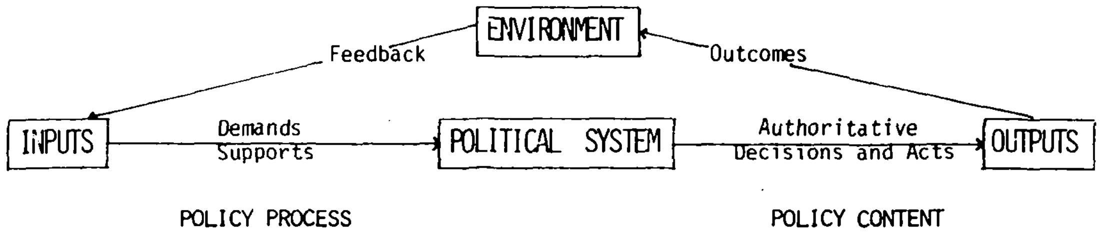

Figure 1. *"Essentials of a Political System"; adapted from David Easton,* A System Analysis of Political Life, New York, J. Wiley & Sons 1965.

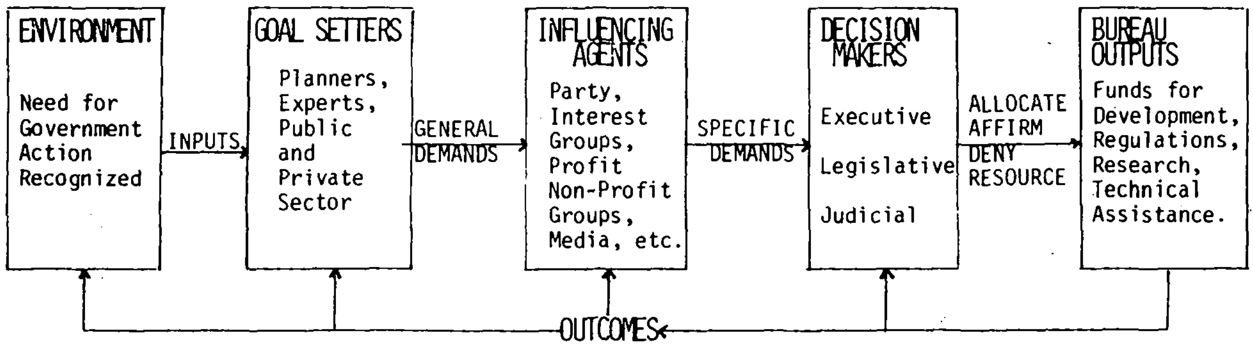

Figure 2. Policy Subsystem Model

ENVIRONMENT Defined Needs - Impacts of Government Decisions	GOALS SET	INFLUENCING AGENTS	DECISION BY GOVERNMENT	BUREAU OUTPUT International Boundary and Water Commission
I 1961-2 Salinity increase in Colorado River waters received by Mexico results in crop losses in Mexicali Valley	Mexico - Reduce salinity to acceptable level based on content of water above Wellton-Mohawk Drain (Imperial Dam) U.S. - *Maintain as in past;* quality not an issue	Mexico - Activist group formed in Valley; demonstrations. U.S - Colorado River basin states reactivate Committee of 14 active in adoption of 1944 treaty.	Executive Only Mexico - President places high priority; formal protest to State Department, treaty has been broken. U.S. State Department denies treaty broken; but Pres. Kennedy and Mexican President reach joint decision, issue communique	IBWC asked to respond with plan in 45 days Study and recommendation
II 1967-72 Temporary outputs by *IBWC* - salinity stabilized at 1250 ppm	Mexico - retains same goals U.S. - proposes "salt balance" criterion - rejected by Mexico	Mexico - post 1964 - no local demonstrations; recur in 1970-2 U.S. Committee of 14 active *throughout to influence* President and Congress	Executive Only Mexico - Remains a high priority with President; visits U.S., speaks to joint session of Congress *1972* U.S. - Supports resolution but no decision is reached until President Nixon appoints the Brownell Task Force	Minute 418 - U.S. construct by-pass; increase releases to dilute *Minute 241* temporary resolution pending adoption of final
III 1973-74 Salinity still stabilized	Mexico and U.S. agree that salinity be reduced Goals set by Minute 242 - 1150 ppm ± 30 ppm	Mexico - unfulfilled goals remain an issue for Mexicali Valley - new dem*onstrations 1971-2 prior* to Mexican President's visit to U.S. U.S. - Committee of 14 supports construction of desalting plant and reduction salinity Colorado above Hoover Dam	Executive & Legislative Mexico - accepts Minute 242 President & Congress U.S. - Presidential low-cost bill replaced by one from basin states; Congress ratifies 242 and passes Colorado River Basin Salinity Control Act to fund commitment; Bu. Rec. not State to undertake desalting commitment	Final Minute 242 Build desalting plant and other *facilities funded by* federal government; additional studies authorized; U.S. to help Mexico get funding for remedial projects.
IV 1984 Goals set by Minute 242 not reached - many parts agreement not fulfilled				

Figure 3. POLICY SUBSYSTEM
U.S. - MEXICO NEGOTIATIONS OVER COLORADO RIVER SALINITY

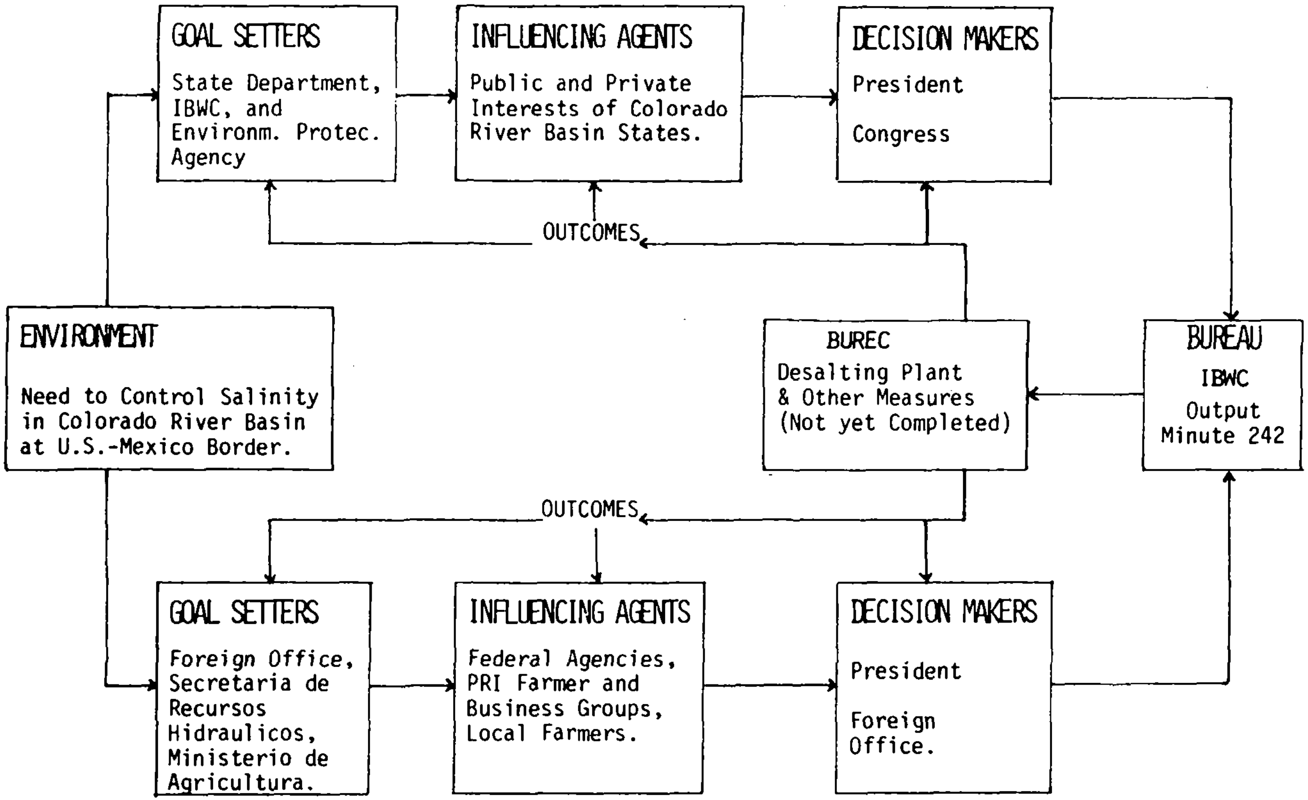

Figure 4. Model Applied to Colorado River Basin Salinity Negotiations Between U.S. and Mexico.

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ECONOMIC, SOCIAL AND TECHNICAL CONSIDERATIONS DETERMINING INVESTMENTS IN GROUNDWATER IN BANGLADESH

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ABSTRACT

With a total land area the size of the state of Illinois and a population exceeding 90 million, Bangladesh has one of the highest man-land ratios in the world. Blessed with vast river systems but having such a flat topography that large-scale reservoir and gravity surface irrigation systems are not feasible, Bangladesh has been forced to turn to groundwater as a source for dry season irrigation water. Initial investmens were in low-lift pumps but now the Government of Bangladesh (GOB) is encouraging investment in hand-pumps, shallow tubewells and deep tubewells as sources for additional water for irrigation. However, to date utilization rates have been far below those predicted by national planners. The purpose of this paper is to analyze the economics of alternative groundwater extraction devices in Bangladesh and to use their results to explain present low utilization rates. Using recent data the analysis examines economic, social and technical characteristics of the alternative technologies and explains why shallow tubewells are to be encouraged over deep tubewells. Based on these results suggestions for improving utilization rates are presented.

Keywords: Groundwater, deep tubewells, shallow tubewells, conveyance losses, water users associations.

INTRODUCTION

In recent years Bangladesh has experienced rapid growth in the minor irrigation sector which is usually taken to include traditional water lifting devices, manually-operated tubewells (MTW's), shallow tubewells (STW's), deep tubewells (DTW's) and low lift pumps (LLP's). During the 1950's and 1960's emphasis was placed on traditional water lifting devices and LLP's, but with the full development of available surface supplies investment strategies turned toward MTW's, STW's, and DTW's. Between 1970 and 1984, Bangladesh installed sufficient tubewells to irrigate over 2.5 million acres annually [World Bank, 1982].

Given high birth rates, declining death rates, and increasing food imports Bangladesh has to continue to increase cropping intensity. Investment in minor irrigation is a major element in this process. However, while there is considerable agreement in Bangladesh that investment in minor irrigation is needed, there is wide disagreement about what form that investment should take. A major factor leading to that disagreement in the recognition that current tubewell technology is not performing as well as expected. In particular there is concern about the low level of utilization of existing irrigation equipment. A second concern is to insure that the mix of equipment installed is both technically and socially efficient.

The purpose of this paper is to analyze the economics of alternative groundwater extraction devices in Bangladesh in order to explain low utilization rates. Recent data from Bangladesh are used in order to incorporate both economic efficiency and social considerations. The final section discusses policy implications and future directions for tubewell development.

Groundwater Development

Bangladesh is predominately an agricultural country with 55% of GDP, 80% of employment and 90% of exports coming from the agricultural sector. With a population of 93 million and a maximum cultivatible acreage of 23 million acres, Bangladesh has a land-man ratio of 0.25, one of the lowest in the world. Possessing large areas of fertile, deltaic soils with temperatures suitable for year-round cropping Bangladesh has the potential for multiple cropping. However, annual rainfall, which varies from 60 to 120 inches, falls mainly during the monsoon season (May to September) and, therefore, during the rest of the year irrigation is required for successful crop production. Fortunately, Bangladesh has substantial groundwater and surface water resources, although surface water potential is almost fully developed.

In order to develop its groundwater resources Bangladesh initially depended upon manual power. However, starting in the early 1960's what is now the Bangladesh Agricultural Development Corporation (BADC) purchased and rented out a small number of LLP's; by 1965 the number of such pumps had increased to over 3,000. Shallow tubewell development started in the 1970's and spread rapidly. As of early 1984 over 120,000 STW's had been installed and were irrigating 1.6 million acres. The total number of DTW's sunk under various schemes is about 18,000 with around 16,500 commissioned as of January, 1984. By early 1984 approximately 220,000 MTW's had been purchased and installed and were serving in excess of 80,000 acres. Under the Medium-Term Food Production Plan (MTFPP), which is designed to attain food self-sufficiency by 1985, Bangladesh plans to increase land irrigated by merchanized lift devices from a 1979-80 level of 2.25 million to 4.76 million acres by 1984-85 [Planning Commission, 1980]. As illustrated in Table 1, this plan involves a significant increase in the number of tubewells (of all types) in use in the country.

With a wide variety of physical, economic, and social conditions no single lifting device is superior in every area. However, from Table 1 in terms of total area served LLP's, STW's, and DTW's are by far the most important mechanical lifting devices in Bangladesh with STW's and DTW's being of immediate concern for groundwater development. Therefore, the remainder of this paper concentrates on these two lifting devices.

Table 1: Present and Projected Status of Mechanical Irrigation Lift Devices in Bangladesh

Device	Approximate Number in Operation (000)		Approximate Area Irrigated (000 acres)		Additional No. Proposed under MTFPP (000)	Approximate Area Proposed to be Irrigated (000 acres)
	1980-81*	1983-84**	1980-81*	1983-84**	1984-85	1984-85
LLP	36.0	36.0	1400	1400	51.0	2000
DTW	11.5	16.2	640	960	18.0	1080
STW	24.0	120.0	270	1600	90.0	1080
MTW	100.0	200.0	40	80	180.0	72
TOTAL			2350	4040		4232

* World Bank (1982).
** Estimated by IADS Water Management Office

Utilization of Irrigation Equipment

Except for MTW's all of the mechanical lifting devices have been sold with a subsidy. These subsidies have been particularly large in the case of LLP's and DTW's. In the late 1970's, BADC accounts indicated that actual payments by farmers for LLP's amounted to 12% of the cost to BADC while for DTW's the figure was less than 10%. Current arrangements for selling tubewells (compared to past policies of renting them) result in prices for STW's that are nominally unsubsidized. However, DTW's continue to be heavily subsidized, with a selling price amounting to only 43% of the cost to BADC [World Bank, 1978].

In addition to the nominal price of tubewells, provisions for the sale of all tubewells involve subsidized credit arrangements which further reduce the effective price paid. For example, for a STW sold for Tk 30,000 (in 1984 23 Taka = $1.00 U.S.) a farmer only has to pay Tk 2,000 as a down payment, and receives a 6-year loan for the balance, paying a reduced rate (12 or 13% depending upon the source of the load), of interest per year. Furthermore, repayment of agriculutral credit has generally not been satisfactory, with repayment rates of less than 70% [World Bank, 1983].

Recent work by Bhuiyan [1984] indicates that present command areas, averaging about 20 acres per one cubic foot per second (cusec)of discharge, are far below physical limits of the technologies used. (Technical potential is estimated by different individuals to vary from 30 to 50 acres per cusec.) As documented in Table 2, there are potentially substantial per acre returns to be generated if command areas can be increased. However, as argued by Small [1983] there is no more reason to presume it is economically desirable to utilize equipment to its technical capacity, than to presume it is economic to utilize land to its agronomic capacity. Just as there are costs associated with achieving maximum yields which may be greater than the value of the increased yield, so too there are costs associated with achieving maximum utilization of equipment which may be greater than the benefits of increased utilization. Thus, the question of the optimal level of utilization of irrigation equipment is ultimately an economic question, and needs to be framed in economic rather than purely technical terms.

Farmers are utilizing irrigation equipment at levels that are considerably below the expectations of those involved in national irrigation planning. This suggests that under current farm-level conditions there is a divergence so that what is optimal for farmers is much less than what the planners had believed would or should be the case.

Table 2: Effects of Increasing Command Area Size on Per Acre Net Present Economic Values (NPEV) Associated With Selected Irrigation Technologies[1]

(in Taka and Percent)

	NPEV With			Change			
	Low[2] Coverage	Medium Coverage	High Coverage	Low to Medium Absolute	Low to Medium Percent	Medium to High Absolute	Medium to High Percent
Deep Tube-wells	2306	3909	4616	1630	71	707	18
Shallow Tube-wells	4389	6049	6916	1660	38	867	14
Low-Lift Pumps (1 cusec)	7954	8870	9273	916	12	403	5
Low-Lift Pumps (2 cusec)	8141	8960	9335	819	10	375	4

Source: Hanratty (1983)

Notes: [1] Average net present economic value across land types.

[2] Low, medium and high coverage (in acres) varies by technology: with DTWs = 40, 60 and 80A; with STWs = 10, 15 and 20A; with LLP (cusec) = 20, 30 and 40A, and; with LLP (2 cusec) = 40, 60 and 80A.

Impacts of Subsidies

Probably the most obvious distortion that leads to a possible divergence between farm-level and national-level optimums is the distortion of prices resulting from the various subsidies on the STW's and DTW's. As illustrated in Figures 1 and 2, the higher the rate of subsidy the smaller the command area required to reach a point where there is no further cost advantage to increased utilization of the tubewell. This is particularly obvious in the case of DTW's under annual rental programs. After 40 acres there is effectively, no incentive to expanding the command area as the rental subsidy has significantly reduced the benefits of spreading capital costs over a larger number of acres.

Conveyance Efficiency

Compounding the impact of subsidies are the additional distributional losses resulting from expansion of the command area, and consequent utilization of a large network of water channels. These losses vary by location, by soil type and by degree of maintenance, but research has shown that the losses are in almost all cases a function of length of channel from the water source [Karim, et al., 1983]. As illustrated in Figures 1 and 2 when conveyance losses are taken into account, the results also serve to discourage expansion of command area.

Social Factors

Two major social factors impact on the costs of water and, therefore, on the area commanded by a tubewell. As pointed out by Hamid [1982], a critical consideration is the pressure that is brought to bear on concerned officials to control the location of the tubewell. In the case of the DTW this usually results in the well being located near the house or fields of an influential farmer. Data from 170 2-cusec DTW's collected in Bangladesh in 1984 indicates that mislocation of these tubewells reduced total potential command area by an average of 11 acres [Johnson, 1983]. Assuming data from these 170 DTW's are representative, it is apparent that improper siting makes rules-of-thumb (such as 40 acres/cusec) difficult, if not impossible, to achieve.

Another consideration in explaining low utilization of irrigation equipment is the cost of operating and managing a tubewell. There are, no doubt, very real "transaction costs" related to managing an expanded command area. With farm size of less than an acre, expanding the area requires communicating with and satisfying a larger number of users. As the area of land served and number of users increases management effort and potential for conflict also increases. These costs are usually not included yet they are costs and have a significant impact on the area commanded by a tubewell.

Hanratty [1983] provides details on DTW management that can be used to estimate per acre organization costs. Based on this information, it appears that organization costs per acre commanded decline until approximately 50 acres are served and then start to increase as the command area expands beyond 50 acres. The situation for STW's is different but the risk of loss due to mechanical breakdowns (which are directly related to hours pumped) argues for keeping the area served less than would otherwise be technically feasible.

DISCUSSION

Both DTW's and STW's were promoted with the assumption that farmers would form cohesive groups to use properly the water-lifting equipment made available to them virtually free of costs. Low utilization rates, particularly for DTW's, indicates that this assumption did not work satisfactorily. Reasons for low utilization of DTW capacity are rooted in technical problems of suboptimal siting, defective water conveyance systems, organizational and management problems of tubewell committees and lack of economic incentives due to extensive subsidies. On the other hand, STW's require less capital and less organizational cooperation, although a cooperative environment is often an economic necessity for raising the capital to purchase a STW.

A question that is often asked is: What type of tubewell technology is the most relevant and appropriate for Bangladesh today? This question can be answered from different viewpoints, but in terms of technology all of the different options DTW's, STW's, LLP's and MTW's are relevant. In areas where shallow aquifers are not available, DTW's are a relevant technological option. For geohydrologically suitable areas, STW's are preferable and can even serve as technological advances for farmers that first start with MTW's.

From Figures 1 and 2 it is apparent that per acre water costs for STW's (Tk 1400-1700) are far less than per acre water costs for DTW's (Tk 3000-4000), unless DTW's are provided for a nominal rental charge as they were in the past. If these figures are approximately correct, it can logically be argued that no DTW's should be installed unless DTW's are the only technological option. Thus, the new Five Year Plan's reduced emphasis on DTW's and increased emphasis on STW's and MTW's is a sound policy.

However, even with the STW's, and particularly with the DTW's, there is a critical need to expand the utilization rate. In order to do this it is necessary to encourage such actions as:

a. removal of subsidies particularly on DTW's but also on STW's as this provides a major incentive to expand commanded area.

b. exploring means of reducing conveyance losses such as lining and use of buried pipe

c. expanding programs such as the Deep Tubewell Irrigation and Credit Program (DTICP) and the Irrigation Management Program (IMP) which have by absorbing some of the "transaction costs", demonstrated another means of expanding utilization.

d. ensuring proper siting of the tubewell.

e. development of cohesive water user organizations to provide proper system management and maintenance.

Together these actions will significantly increase returns to irrigation investment and therefore facilitate additional groundwater development.

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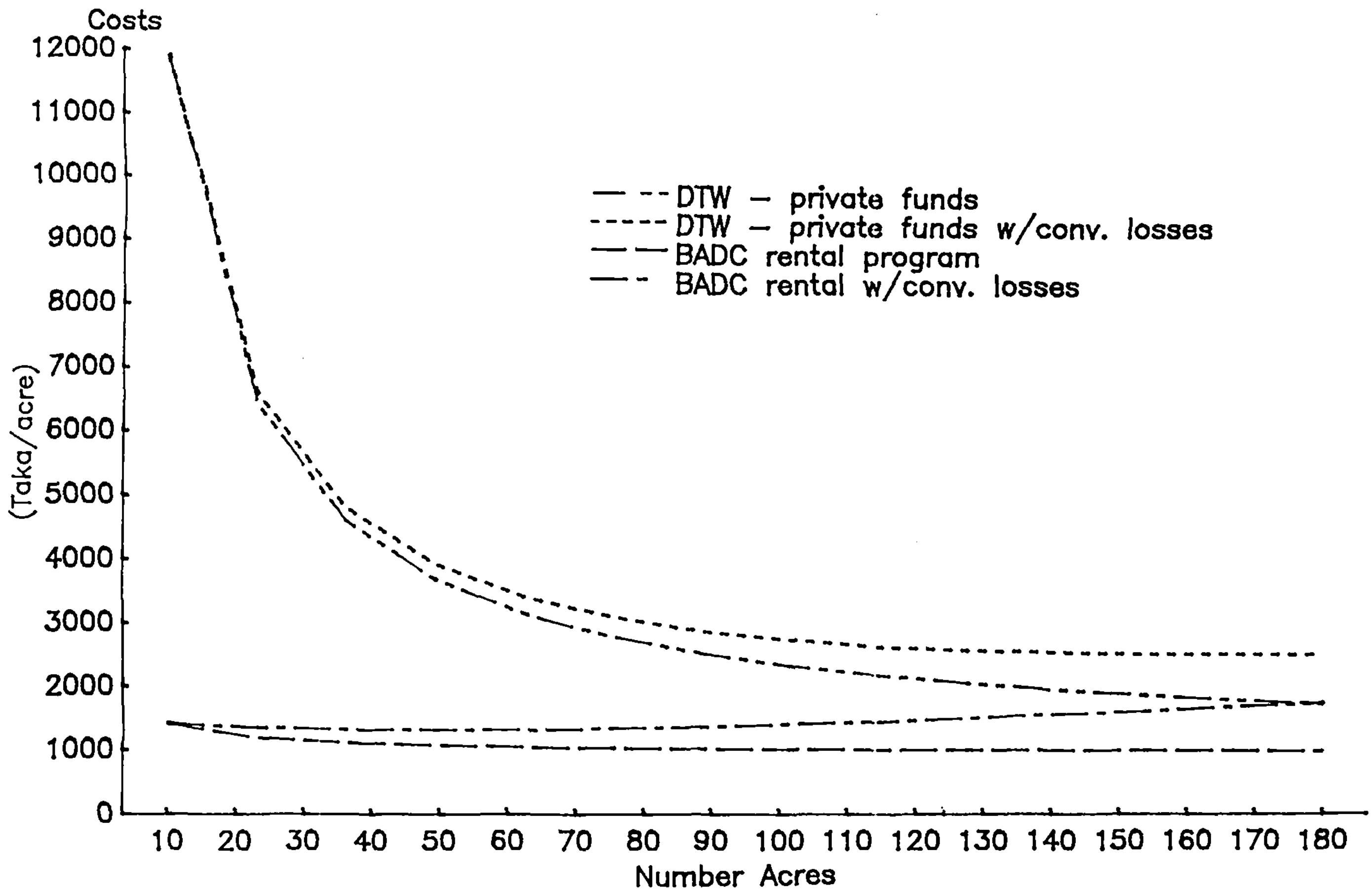

Fig. 1. Costs of DTW water with and without conveyance losses.

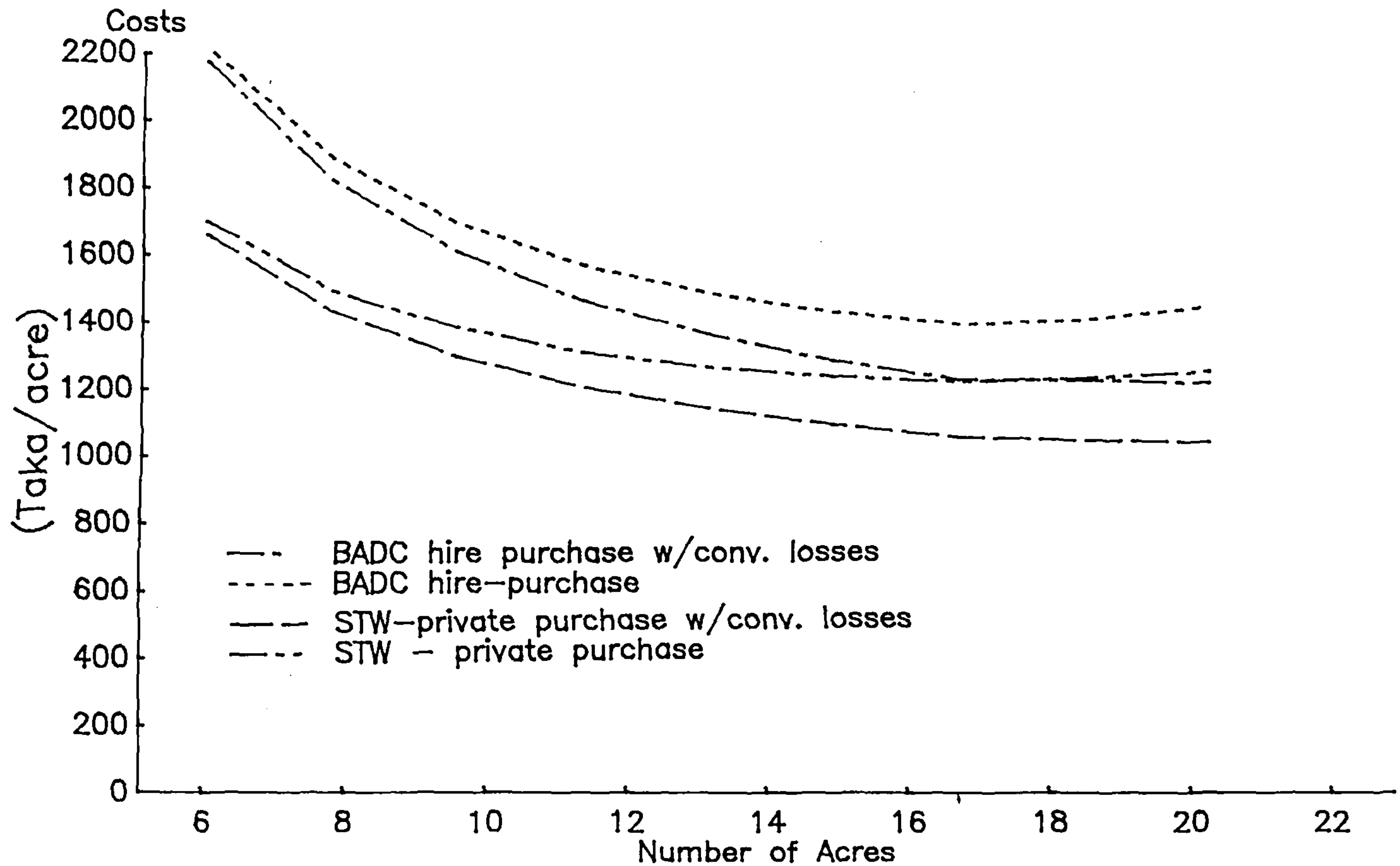

Fig. 2. Costs of STW water with and without conveyance losses.

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 237 Aspect number 6

THE DEVELOPMENT OF WATER RESOURCES THROUGH FRESHENING RESERVOIR AT POLLUTED SALINE WATER REGION

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ABSTRACT

Topographically the standard method which constructs dam in mountain area and long canal system in plain is especially too expensive to get water resources for delta region in present. The freshening reservoir method is some time very effective one which can develop the low cost rural water resources without the disturbance of water use right occupied already by the people in the region of upstream area. But the problem which should be solved integratedly in planning becomes more difficult than standard method.

The special important items to be solved at polluted saline water region are;

1) freshening reservoir should satisfy the balance of water quantity not only for water utilization but also for desalinization release.
2) freshening reservoir should control salinity concentration to the limitation of paddy rice, upland crops and drinking use.
3) freshening reservoir should control the water quality which is one important factor of environmental problems through the improvement of hydraulic structure of reservoir and the cutting of pollutant matter from basin.

The Nakawmi freshening reservoir project which lies in Shimane prefecture Japan, has very serious situation and the counter measures could be adopted here systematically through the reasonable research works and the improvement of hydraulic structure.

The development of saline water region would be very important as a rural area water resources in 21 countries.

Keywords :

freshening reservoir rural water resources
water quality turbulent density flow

1. The outline of present situation and freshening planning of Nakawmi lake and Shinjiko lake of saline water region

Usually in humid Asia, for example in Japan, the rate of utilization of river water to the total run off is less than 15%, and great quantity of river water is going to the sea uselessly in rainy season.

The water resources development by freshening reservoir method which has low water cost can be introduced here as a new water resources of deltaic area. But we must solve the complicate difficult problems in planning of water resources development comparing the standard method in mountain area. The freshening reservoir includes generally complicate environmental problem because the river is receiving all disposal from upstream towns and agricultural lands. Also such project site lies usually in saline water polluted area connecting estuary and sea.

The Nakawmi freshening reservoir Project is one example of this kind of special development.

The important results in above mentioned research work is that the water quality in N-lake can be improved through reasonable design of outlet gate and selective bottom water release facility and the cut of secondary circulation of pollutant matter accompanied with sea water back flow with density current into N-lake.

Fig.1 shows the system of flow including Hii river, Shinjiko lake and N-lake. In the Figure field survey points of water quality are marked from No.1 to No.23. The sampling of water is done continuously once a month every year so far.

Table 1 shows the comparison of S-lake and N-lake between present situation and plan. The water surface of N-lake will decrease from 97 km^2 to 66 km^2 and storaged volume of N-lake will decrease from 5.2 * 10^8 to 3.7 10^8 m^3 by reclamation works. But the annual inflow is kept as constant 2.5 * 10^8 m^3/year.

The Cl'-concentration will decrease by freshening works from 2,000 ppm to 200 ppm in S-lake, and 12,000 ppm to 200 ppm in N-lake. There are serious competition on environmental problem and various opinion occurs during several years.

2. Present hydraulic structure of the Nakawmi lake

The present flow system including N-lake can be shown in Fig.1, namely there is the water flow system from Hii river to Japan sea. Among two boundary points, S-lake, Ohashi channel, N-lake and Sakai channel are continuously included.
The salinity of each area is as follow.

Hii river	fresh water discharge
⇓	
Shinjiko lake	blackish water
⇓	
Ohashi channel	blackish water
⇓	
Nakawmi lake	blackish water
⇓	
Sakai channel	blackish and sea water
⇓	
Miho bay (Japan Sea)	sea water

As the S-lake lies in upstream of N-lake, so the main area to improve water quality is the N-lake which is getting more saline water from sea and more pollutant matter from entire basin.

2-1 hydrological situation of the flow system

(1) Water elevation

The mean water elevation at S-lake is + 0.1m, N-lake + 0.05m and the

Japan sea 0.0m. The tide of Japan sea is very small, 0.2 $\sim$ 0.3 m amplitude.

(2) discharge

The discharge of entire basin is going through Hii river, S-lake, Ohashi channel, N-lake and Sakai channel to Japan sea with about 70 $\sim$ 80m^3/s averaged in a year. The annual run off is 25 $*$ 10^8 m^3/year. The alternative flow due to tidal phenomenon, namely 20 $*$ 10^6 m^3 of sea water, is going and back during one period of the tide in Sakai channel. But it is very important that the actual sea water and the polluted water in N-lake can not be exchanged directly by the effect of tidal flow because the distance of go and back of water mass during one period is less than several km, and this is shorter than the length of Sakai channel 9 km.

2-2 hydraulic situation (turbulent density flow)

There is two layered flow in Sakai channel and N-lake. River water is going to sea through upper layer and sea water coming back through lower layer of Sakai channel to N-lake. The special hydraulic structure in Sakai channel and N-lake depends on the turbulent density current. This density current plays very important role for the pollution mechanism in N-lake producing the secondary circulation as shown in Fig.2.

2-3 The estimation of sea water back flow into the Nakawmi lake

All salinity in S-lake and N-lake might come from Japan Sea through bottom with sea water as a density current. The approximate discharge of sea water back flow in N-lake and S-lake in average in a year can be estimated as about 70 m^3/s.

2-4 The hydraulic mechanism of pollution supply accompanied with sea water back flow into Nakawmi lake

Basically there are special hydraulic structure which has close relation with water pollution in N-lake. Namely, the upper layer waters in N-lake and Sakai channel are flowing from Ohashi river to Japan sea. The lower layer water is flowing from sea to upstream through the bottom of Sakai channel and N-lake, and small water mass will be exchanged between both layers. The relation can be shown in Fig.2. The salt water in region No.2 in N-lake had nearly the same hydraulic structure with that of Sakai channel between upper layer and lower layer. In the region 4, the strong vertical mixing between both layers can be generated, and two layered density current become intensive mixing type and the difference of salinity between both layers becomes very small.

Writer could deduce the averaged vertical mixing model as show in Fig.7 in time, and distance. Combining this averaged vertical mixing velocity and both flows in upper layer and lower layer the secondary circulation model could be deduced as shown in Fig.2.

(i) Field survey on hydraulic structure of Nakawmi lake

To make clear the special hydraulic structure with two layered turbulent density current due to salinity distribution, we performed longitudinal field survey at many points in N-lake in Fig.3, with one km interval in horizon and each 0,5 m in depth for the salinity (electricity), disolved oxigen and water temperature. The results are as shown in Fig.4, 5, and 6.

2-5 vertical hydraulic stability of Nakawmi lake and Sakai channel

Usually the two layered density stratified current is generated by river water with lower density in upper and sea water layer with higher density coming from Japan sea through the bottom of Sakai channel and N-lake.

The mean velocity of horizontal current in both layers are comparatively small which are generated by river discharge, tidal current and wind driven current. Fig.8 shows the vertical distribution of Cl-ion.

Then the hydraulic stability of N-lake is very high as we can guess through the calculation of Richardson Number.
This high hydraulic stability is creating various kinds of bad effect to water quality in N-lake. Namely the steep slope of density distribution is depressing the vertical mixing velocity completely.

3. The present level of Water pollution of the Nakawmi lake

Though there are two lakes, S-lake and N-lake, the downstream N-lake is more serious for water pollution. The present situation of water pollution of N-lake can be understood by the data observed, TP ≒ 0.2 PPm in lower layer and TP ≑ 0.007 in upper layer in summer.

The origin of the present pollution of the N-lake are classified in three items.

(1) inflow of polluted matter from the basin

As the basin of N-lake has the basic data of pollutant shown in Table 2, so the inflow of total phosphorous into the N-lake is estimated as about 6 gr/sec. (There is the new plan of sewage treatment by the Shimane-Prefecture.)

(2) water pollution due to hydraulic stability in Nakawmi lake

The various situation of water pollution can be deduced through these hydraulic stability as follows.

(a) Salinity distribution

Fig.10 shows the longitudinal distribution of salinity in both layers, upper layer and lower layer, from the downstream end (two kilometer upward from Japan sea) to S-lake through Sakai channel, N-lake and Ohashi channel, and Fig.8 vertical distribution.

The special character is that the sea water is coming back from Japan sea to the N-lake and to Yonago bay with small vertical mixing in vertical, and almost of salt water is completely mixed at the salt wedge front covering from Ohashi channel to the Yonago bay which has internal boundary layer about 3 meters depth.

(b) dissolved oxigen

Figs.11 shows the longitudinal distribution of dissolved oxigen in both layers, and Fig.9 the vertical distribution. The salinity spring layer has completely depressed the vertical transfer of oxigen from water surface to lower depth.

(c) Total phosphorous

Fig.12 shows the longitudinal distribution of total phosphorous in both layers. In the Sakai channel, the concentration TP is low in lower layer than upper layer due to the sea water back flow, but in the N-lake region, the concentration of TP in lower layer become very high than upper layer contradictory to the case of Sakai channel. This phenomenon should be understood considering secondary circulation in Sakai channel and N-lake, and dissolved PO_4 from the bad soil.

(3) Secondary circulation of polluted matter in the Sakai channel and Nakawmi lake

The polluted matter, for example total phosphorous are flowing in upper layer from upstream to the sea and a part of this is flowing back accompanied with bottom density current receiving the pollutant matter by vertical mixing velocity. The mixing velocity and two layered density flow constructs the secondary circulation as shown in Fig.13. The secondary circulation in Fig.13 will play some special behaviour, namely much TP is mixed into upper layer from lower layer which has highest TP concentration at No.3 and No.4 at the salt wedge front in N-lake.

The dissolved PO_4 from bed soil can be controlled by the concentration of dissolved oxigen on the bottom. When there are no oxigen, much PO_4 can be dissolved but if there are much oxigen, the dissolved PO_4 from the bottom soil can be completely depressed. Also the dissolved PO_4 is conveyed with sea water back flow in N-lake. The total phosphorous conveyed by sea water back flow can be estimated as shown in Fig.19.

4. The planning of freshening reservoir

For the purpose of water resource development and improvement of water quality the following freshening reservoir is constructed.

(1) facility for freshening

(i) outlet gate

The facility of freshening reservoir in N-lake consists of Nakawra outlet gate with three lock gates, desalinization facility and communication bridge as shown in Fig.14. Fig.15 shows the detailed plan of desalinization facility. The descriptions are written in Table 3.
The outlet gate can control the depth of internal salt spring layer in reservoir, from -(3 ~ 4) m to - 7.0 m easily.

(ii) desalinization facility

The desalinization facility system is constructed in the N-lake. There is the bottom canal excavated under the bottom of lake from the outlet gate to Yonago bay where is the most polluted part in N-lake. At the down stream end of the bottom canal, the salt water pocket which has the depth - 16 m and area 50 ha is excavated. The salt water in this pocket can be released through bottom drainage pump to Sakai channel. Through the reasonable operation of these desalinization facility, we can control the salt spring layer from - 7.0 m to deeper depth - 9.0 m. By this control, all salt water in N-lake can be easily drained. The pollutant matter which has high concentration in deep part, TP and TN, can be also drained accompanied with such salt water from the pocket.

(2) Estimated freshening Process

Freshening process of the Nakawmi freshening reservoir can be shown by Fig.16.

If we adopt only the outlet gate method the process will delayed seriously as shown by full line in Fig.16, and we can not get the water less than 1,000 ppm Cl' in terminal situation. We adopted the special desalinization system as above mentioned, then freshening process is accelerated very much and the terminal salinity concentration will reach less than 200 ppm Cl' within about two years.

(3) improvement of hydraulic structure by the construction of Nakawmi freshening reservoir

The removement of stratified two layered zone in N-freshening reservoir has created very important function to improve and control the water quality of N-freshening reservoir. The change of flow pattern by freshing wooks can be compared between Fig.17 in present and Fig.18 after the construction of N-freshening reservoir. Namely, the freshening works will diminish the density current with stratified two layers and uniform current in vertical can be created.

5. Conclusion

When the freshening operation has started, we will get the result of water quality improvement as follows.

(1) cut off of pollutant matter accompanied with secondary circulation of the sea water back flow

As mentioned in Section 4, the great quantity of total phosphorous which is supplied into N-lake accompanied with secondary circulation due to sea water back flow can be neglected as shown in Fig.19.

(2) improvement of dissolved oxigen

As the salt spring layer could be disappear, so the oxigen in the deep water of N-freshening reservoir will increase drastically.

(3) depress of PO_4 dissolved from bottom soil

The PO_4 dissolved from bed soil can be depressed by the increase of oxigen in deep water.

Also the ecological-system will be changed from blackish one to fresh water. Concerning these problems, we are continuing intensive study now.

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(1) The report on hydraulics, water quality and ecological behaviors relating with freshening reservoirs of Nakawmi and Shinjiko, 1982. Committee of water management and ecological changes by freshening reservoirs of Nakawmi and Shinjiko.

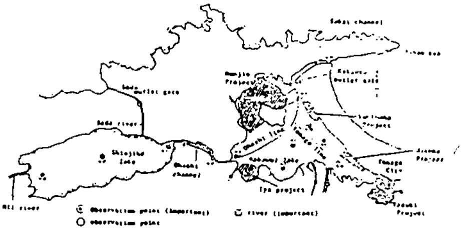

Fig.1 Flow system of Hii river, Shinjiko lake, Ohashi channel, Nakaumi lake, Sakai channel and Japan sea (1982)

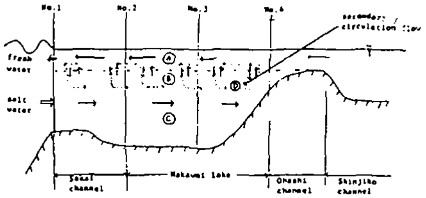

Fig.2 model of main flow in upper fresh water and lower salt water and secondary circulation flow generated with vertical mixing velocity and main flow

No.1 region the boundary between Sakai channel and sea
No.2 region the boundary between N-lake and Sakai channel
No.3 region the middle point in N-lake
No.4 region the boundary between Ohashi river and N-lake
Ⓐ fresh water flow including much pollutant matter
Ⓑ vertical mixing velocity between upper layer and lower layer
Ⓒ sea water back flow from sea to No.4 in N-lake
Ⓓ secondary circulation flow

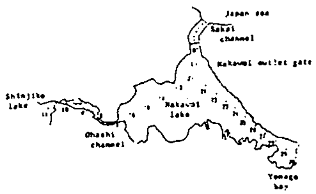

Fig. 3 Distribution of observation point

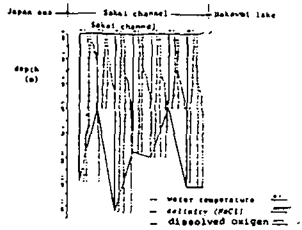

Fig. 4 Longitudinal distribution of the vertical distribution of water temperature, NaCl and dissolved oxigen under 1 km interval (1980)

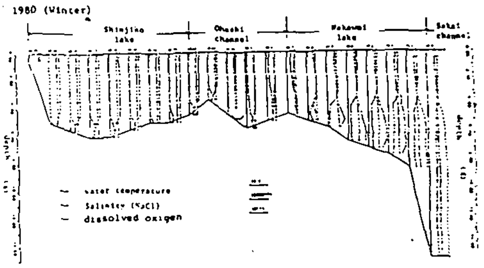

Fig. 5 Longitudinal distribution of the vertical distribution of water temperature, Salinity (NaCl) and dissolved oxigen under 1 km interval (1980)

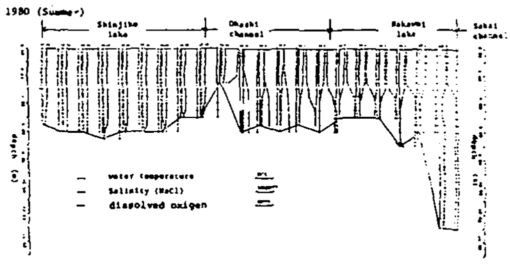

Fig.6 Longitudinal distribution of the vertical distribution of water temperature, Salinity (NaCl) and oxigen under 1 km interval (Summer, 1980)

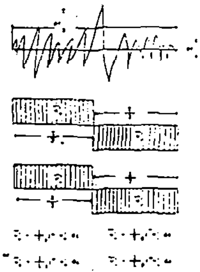

Fig.7 An example of the vertical distribution of horizontal and *vertical fluctuating velocity under the stable density* distribution in experimental canal and mean vertical mixing velocity V'+.V –
where ② 2cm under the water surface
⑱ the center of internal boundary of salinity spring layer

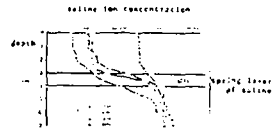

Fig.8 vertical distribution of saline ion at the center of Nakawmi lake (1976)

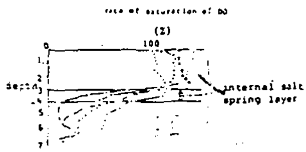

Fig.9 vertical distribution of the rate of saturation of dissolved oxigen at the center of Nakawmi lake (1970)

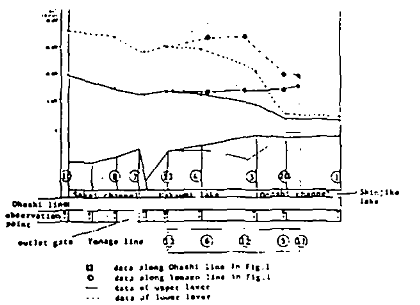

Fig.10 Longitudinal distribution of Cl' concentration in upper layer *and lower layer in Sakai channel* and Nakawmi lake (1980)

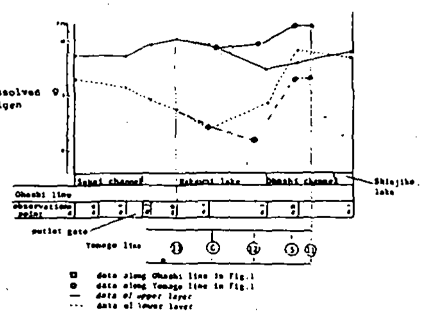

Fig.11 Longitudinal distribution of dissolved oxigen concentration in Sakai channel and Nakawmi lake in Summer (1980)

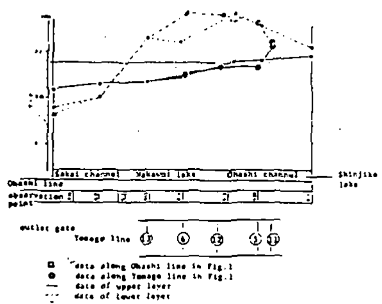

Fig.12 Longitudinal distribution of total phosphorous concentration in Sakai channel and Nakawmi lake in Summer (1980)

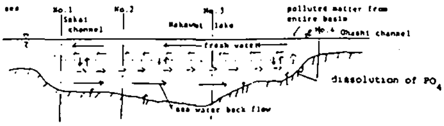

Fig.13 circulation of polluted matter in Sakai channel and Nakawmi lake in present

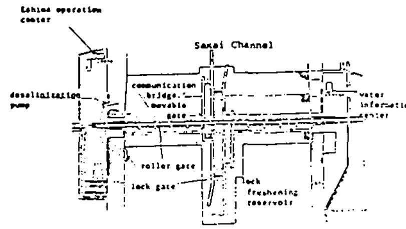

Fig.14 plan of outlet gate

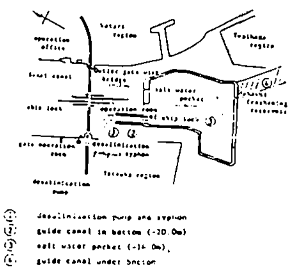

Fig.15 desalinization facility system

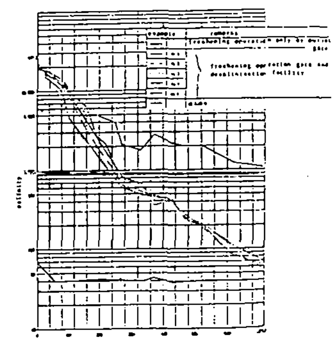

Fig.16 Freshening process graphs

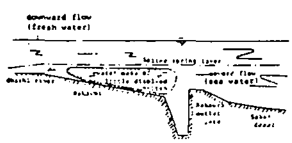

Fig.17 flow model of present N-lake

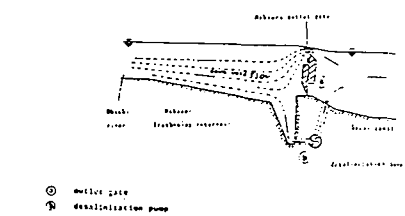

Fig.18 flow pattern in freshening reservoir

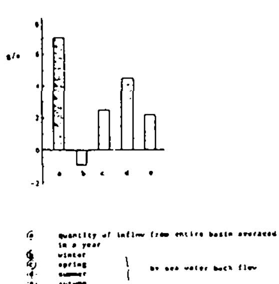

Fig.19 inflow of total phosphorous into Nakawmi lake accompanied with sea water back flow

Table 1. Shinjiko and Nakawmi lakes and its planned freshening reservoir

class	area of water surface plan		area of watershed	area of water surface +area of watershed		concentration of Cl'	
	present	plan		present	plan	present	plan
Nakawmi lake	97	66	644	741	710	12,000	200
Shinjiko lake	81	81	1,227	1,308	1,308	2,000	200
total	178	147	1,871	2,049	2,018	-	-

class	volume		mean depth (maximum)(m)	total inflow in a year (1957) ($\times 10^8 m^3$)
	present	plan		
Nakawmi lake	5.2	3.6	5.4 (8.4)	} 25
Shinjiko lake	3.7	3.7	4.5 (6.4)	
total	8.9	7.5	-	25

Table 2. basic data relating pollution inflow into lakes

item	remarks	
1. basin	2,068 km^2	
2. number of town, village and city	Shimane 22 cities, towns and villages Tottori 2 cities Total 24 cities, towns and villages	
3. area of lake	Nakawmi 96.9 km^2, Shinjiko 80.3 km^2	
4. human activity in basin	pollution industrial products number of animals an area of a paddy field an area of a farm land an area of a mountain. grass land	422,316 people 376,634,000,000 yen 24,177 head of cattle 24,877 head of swine 19,074 ha 3,150 ha 166,856 ha
5. Total inflow of pollutant matters	COD 28.0 t/day T-N 6.6 t/day T-P 0.7 t/day	

Table 3. Outlet gates

	items	Nakawra central gate	Sada outlet gate
outlet gate	total width	434.0 m	50.05 m
	unit span, number	32.0 m 10 gates	19.65 m 2 gate
	sill elevation	(-) 6.80 m	(+) 3.00 m
	elevation of the top of the gate	8.45 m	4.5 m
	type, structure	reinforced concrete structure, double leaf. over flow type roller gate	reinforced concrete structure, roller gate
	bridge	effective width 6.0 m, unit length 35.0 m, elevation of bridge (+) 5.20m	effective width 4.5m unit length 21.63m, elevation of bridge (+) 4.50m
lock gate	number of lock gate	large-size middle-size small-size	one place
	capacity	5,000 weight ton 2,000 weight ton 1,000 weight ton	10 total ton
	width of room	20.0 m 14.0 m 12.0 m	5.0 m
	length of room	150.0 m 103.5 m 70.0 m	22.0 m
	bottom elevation	(-)7.30 m (-)5.50 m (-)5.00 m	(-) 2.30 m
	type of lock gate	steel radial gate	steel roller gate
desalinization facility	desalinization syphon	3 sets, L=186m, B=3m, H=4m, reinforced concrete, box culvert	
	desalinization pump	Q=425.4m^3/min total pump head 1.4m vertical axial flow pump ϕ1,900mm	
	salt water pocket in bottom	bed elevation (-) 16.0 m, area about 25 ha	

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WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 239 Aspect number 3

ADDING AN ENVIRONMENTAL DIMENSION TO COMPREHENSIVE WATER RESOURCES MANAGEMENT IN DEVELOPING COUNTRIES

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ABSTRACT

Following United States experience, developing countries have begun to incorporate natural systems and environmental assessment and valuation techniques in their water resources planning activities.

As a contribution to development of such techniques, an overall framework for natural system assessment and valuation is presented using diagrams of causal and analytical sequences of assessment and the water resources planning process. A more detailed analytical approach to assessment and valuation is shown, starting with a depiction of human activities affecting the natural system, and continuing with quantification of specific natural systems effects using appropriate models. Finally, the problem of economic valuation of natural systems effects and impacts on humans is discussed in the context of evolving techniques of valuation.

INTRODUCTION

Comprehensive water resources planning in the United States evolved from an almost exclusive concern with development in the 1930's and 1940's to a broader multiple-purpose approach in the 1970's and 1980's incorporating concern for environmental quality and sustainable natural systems. An important factor in this evolution was the enactment of the U.S. National Environmental Policy Act (January 1970) with its requirement that an "Environmental Impact Statement" be prepared and submitted along with each water resources project proposal.

This requirement led to the development and testing of a number of methodologies for environmental assessment and valuation of water resources projects and to attempts to integrate them with the water resources planning process (Nichols and Hyman 1982). The U.S. Water Resources Council issued a set of principles and guidelines embodying two primary objectives with corresponding sets of accounts--for environmental quality and national economic development (U.S. Water Resources Council 1983).

As developing countries became increasingly concerned with environmental and natural systems issues in the 1970's, various attempts were made to draw upon United States knowledge and experience with environmental impact assessment to develop methodologies and approaches appropriate to situations in these countries. A number of these were developed for the water resources field (Organization of American States 1978, TAMS 1980, Interim Mekong Committee 1982a, Interim Mekong Committee 1982b).

The East-West Environment and Policy Institute (EAPI), working collaboratively with experts from Asia and the Pacific, has developed an overall approach to natural systems assessment and economic valuation suitable for application in developing countries (Carpenter, editor, 1983; Hufschmidt, _et al_. 1983).

FRAMEWORK FOR ASSESSMENT AND VALUATION

To deal with the problem of how to link natural systems assessment with water resources planning, Figure 1 depicts a framework showing (1) a causal sequence in water resources development, and (2) an analytical sequence in water resources planning (Carpenter 1984).

Looking first at the causal sequence: water resources development _activities_ that use, affect, or are affected by natural systems cause _changes_ in natural systems which in turn have _impacts_ on human health and welfare. The changes and impacts can be positive or negative, beneficial or harmful. For example, a water resources development project for flood control involves a development _activity_, e.g., a flood control reservoir, which _changes_ the natural system, e.g., reduces flood peaks, with _beneficial_ effects on human welfare. Using this causal sequence model allows the planner or analyst to take account of all changes and impacts, both beneficial and harmful.

The analytical sequence part of Figure 1 shows how for each stage the analysis would proceed in terms of identifying, quantifying and monetizing

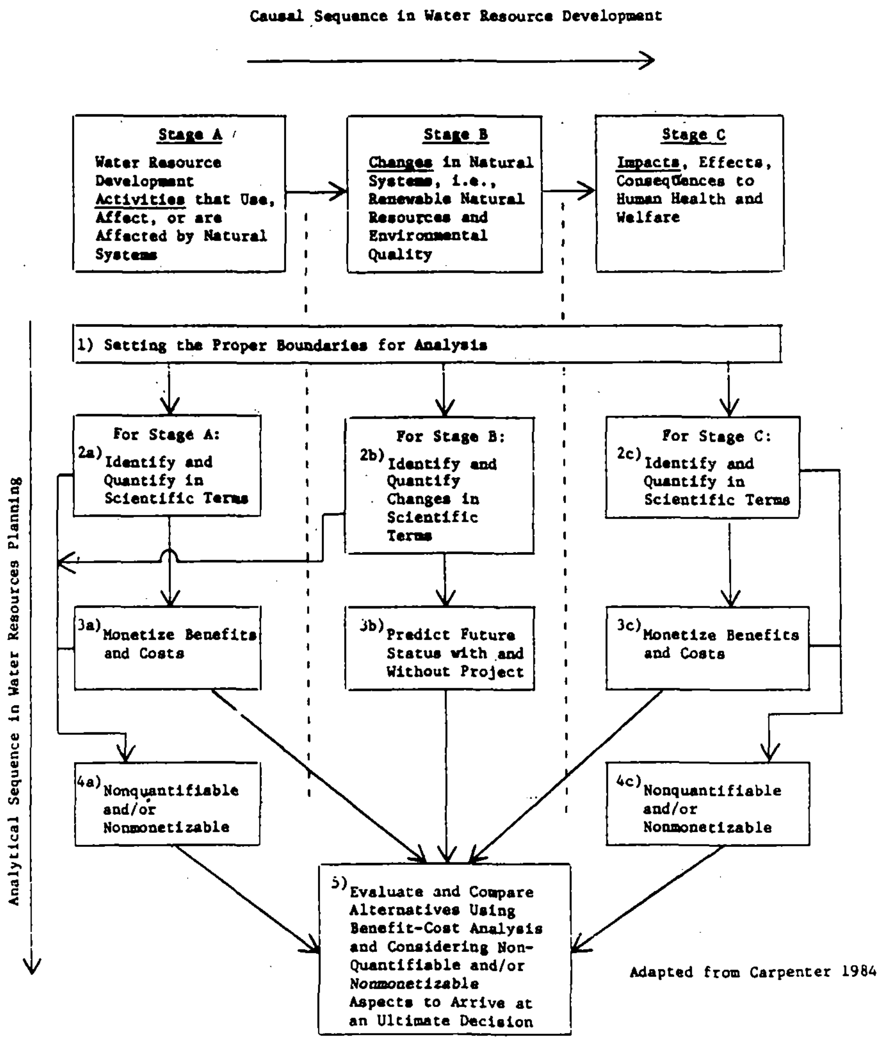

FIGURE 1. Natural Systems Assessment with Economic Valuation of Water Resources Development

activities and effects. For example the changes in natural systems caused by the water resources development activities identified in Stage A would be analyzed in Stage B.

The quantitative information on changes in natural systems can then be used in Stage C to identify, quantify and value the impacts on human health and welfare including economic welfare. Finally, the results of these analyses are brought together in comparison and evaluation of the alternatives, using benefit-cost analysis and the information that was generated on nonquantifiable and nonmonetizable aspects. The results of this summary evaluation of alternatives are presented to decision makers.

The way in which natural systems assessment can be incorporated in the water resources planning process is depicted in Figure 2. At the very first step, the identification and definition of problems, needs and opportunities now includes natural systems aspects such as for problems: water pollution, soil erosion, salinization; for needs: clean water; for opportunities: provision of parks and wilderness areas.

In the second step, the geographic, timing and subject matter scope of the planning will now reflect the entire range of natural system factors that need to be taken into account. Also, a basic natural systems-environmental quality objective will now be added to the other societal objectives, such as national economic development, and the planning guides and criteria will be modified to reflect this change.

The third, plan formulation, step incorporates the analyses shown for Stages A, B, and C in Figure 1. That is, as each alternative plan is formulated, the resulting natural systems changes and their effects on humans are identified, quantified and valued in economic terms, and the total monetary costs and benefits are assembled along with information on nonquantifiable and nonmonetizable aspects, for summary evaluation in the final step of the process.

The final step, review and evaluation of alternative plans, contains all of the relevant information on natural systems and environmental quality consequences of the plans. The ultimate decision that emerges from the decision-making process is made on the basis of balancing the natural systems effects with the economic development consequences.

THE ANALYTICAL APPROACH

Human activities and natural systems effects in a river basin

Taking the river basin or watershed as the relevant land-water system to be examined, human activities that affect natural systems an be listed in two broad classes:

1. activities on land that affect the water resources; and
2. activities involving water resources that affect the land resource as well as water resources.

The advantage of this classification is that it identifies all relevant human activities in terms of uses of land and technologies involving water

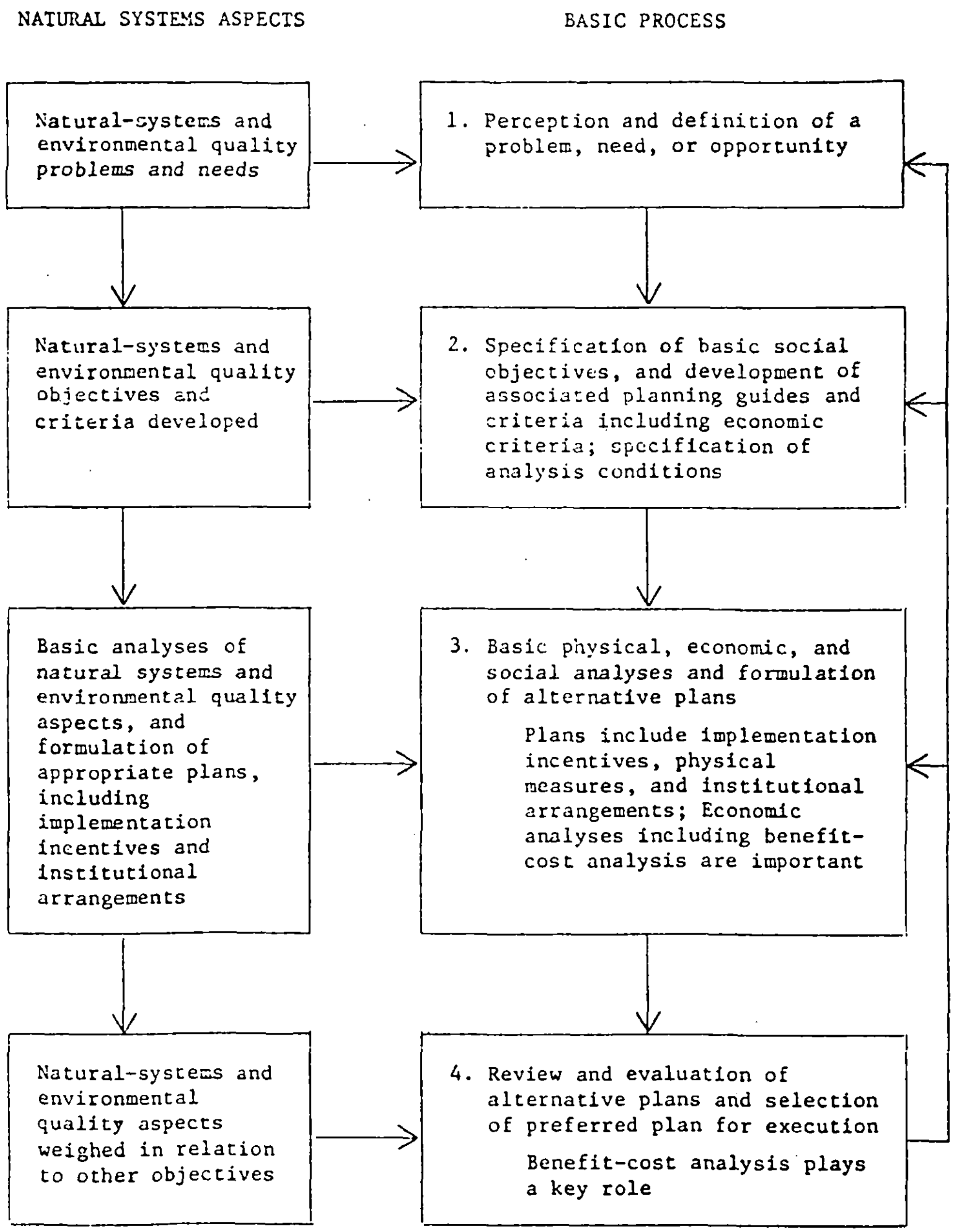

FIGURE 2. Water Resources Planning Process including Natural Systems Assessment

resources. The classification and listing thus provide a useful basis for constructing more detailed checklists, matrices and networks of natural systems effects involving water resources developments.

An important point is that human activities, whether occurring on land or involving water, may have important effects on either or both the land and water portions of natural systems.

Analysis of natural systems effects

To analyze the interactions of human activities and natural systems so that the results can be used in detailed planning of alternatives, it is necessary to use models that portray the physical-biological-chemical processes at work. In the early stages of planning, these models can be relatively simple representations of the links in the causal chain, with indication of direction of effects—increase or decrease. Figure 3 depicts a simple model of this type.

Similar network diagrams, tracing the effects of human activities on elements of the natural system and on other human activities can be developed for other phenomena including flooding, water pollution, waterlogging, and salinization (Hufschmidt, et al 1984, pp. 116 and 137, Organization of American States 1978 Appendix C, pp. 80-88).

Natural systems models

As water resources planning proceeds and specific alternative land use patterns are identified, it becomes necessary to quantify these natural systems effects in order to estimate their impact on human health and welfare. For this purpose it is necessary to use more elaborate models capable of handling cause-effect phenomena in quantitative terms. Useful classifications of natural systems models are contained in Basta and Bower (1982) and Hufschmidt, et al (1983). Most modelling of natural systems uses either (1) the statistical or "black box" approach, (2) the conservation of mass and energy approach, or a combination of them.

The statistical approach views the natural system as a "black box" in which the inputs are independent variables, the outputs are the dependent variables and the internal workings of the black box remain unknown and unspecified. Examples of operational black box models are the Universal Soil Loss Equation and Hydroscience Simplified Water Quality Model (Basta and Bower 1982).

The conservation of mass and energy approach involves developing a set of equations for keeping account of the flow of mass or energy in a natural system. These mass-balance and energy-balance equations indicate that for any size volume in space—in air, water, or soil—the change of mass or energy over any given time must be accounted for by either or both (1) inputs to or outputs from the volume and (2) transformations in the form of mass or energy within the volume. Analyzing a natural system according to these principles entails dividing the system into volume segments and tracing the movement over time of material and energy flows from segment to segment using mass- and energy-balance equations (Hufschmidt, et. al. 1983, Chapter 5).

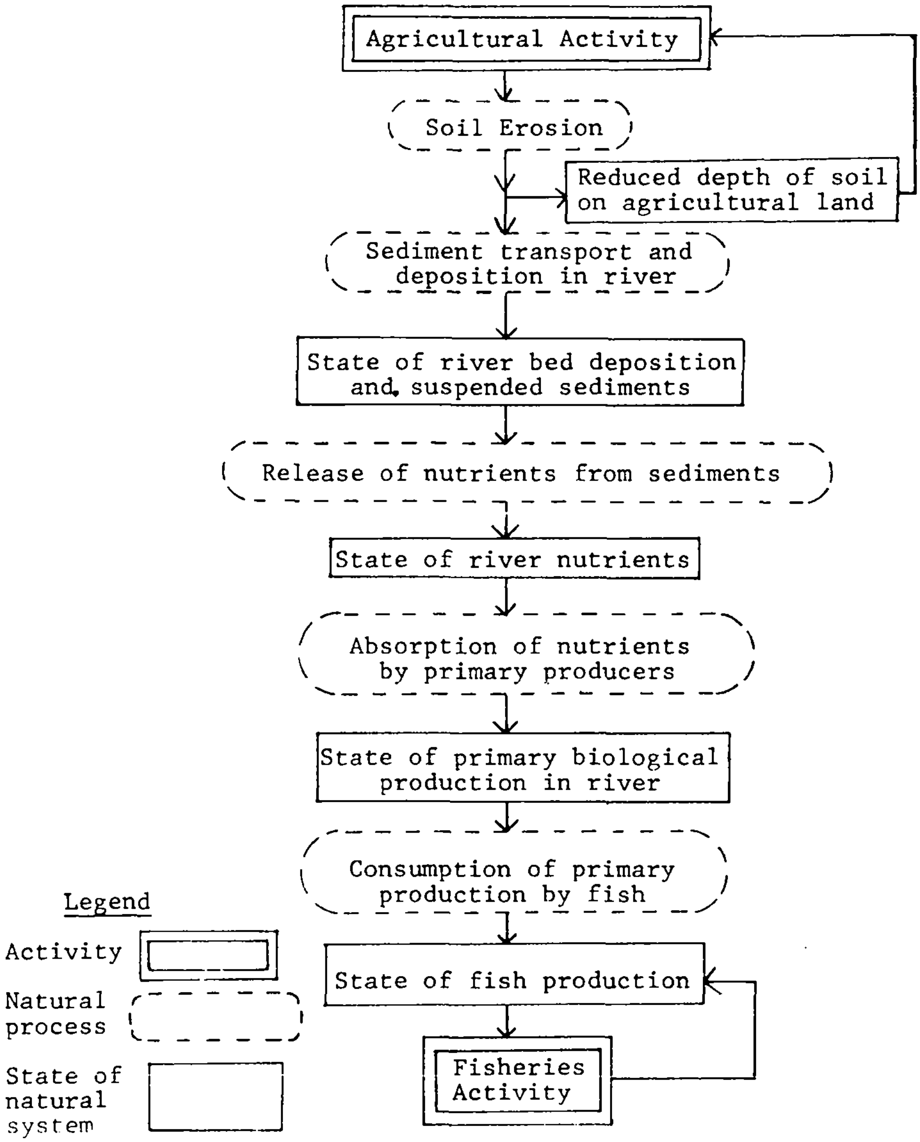

FIGURE 3. Flow Diagram of Effects of Agricultural Activity on the Natural System, including Agricultural Land, Streams and Fisheries

Adapted from Organization of American States (1978)

Important examples of the conservation of mass and energy approach are: Stanford Watershed Model. Streeter-Phelps Dissolved Oxygen Equation, and the Agricultural Runoff Model (Basta and Bower 1982). A comprehensive listing and summary descriptions of natural systems models are contained in Basta and Bower (1982).

The way in which natural systems models can be used in a natural systems analysis is shown in Figure 4, where the effects of erosion from an agricultural activity on stream water quality and fish life are quantified. In this example both statistical and conservation of mass and energy models are used.

Valuation of natural systems changes and impacts

The outputs of the analyses using natural systems models will be quantitative and qualitative information on changes in natural systems and effects of such changes on receptors including humans. The next step is to estimate the significance of these changes and effects in terms of human health and welfare. Benefit-cost analysis is an extremely important technique for measuring the economic dimensions of welfare.

Valuation of natural system effects in monetary terms poses special problems that are beyond those encountered in conventional benefit-cost analysis. These problems arise largely beause usually markets do not exist for the services of natural systems, e.g., the gene pool characteristics of tropical forest, and some of the natural systems goods and services such as air and water quality improvements are "collective," and cannot be exchanged in markets. In spite of these problems, considerable progress has been made. largely in the United States, in developing and applying economic valuation techniques to natural systems effects (Hufschmidt, et. al. 1983). The U.S. Water Resources Council (1983) has codified a number of such techniques in its principles and guidelines for water resources planning and the U.S. Environmental Protection Agency (1983) has been promoting the use of such techniques in its regulatory activities. The U.S. General Accounting Office (1984) has reported that, although major problems still exist in applying economic valuation techniques to natural systems effects of development, the state of the art is sufficiently advanced that benefit-cost analysis has become a useful tool for guiding national decisions concerning natural systems and the environment.

Detailed listing and discussion of these economic valuation techniques are beyond the scope of this paper. An in-depth treatment is contained in Hufschmidt et. al. (1983).

SUMMARY

The major points developed in this paper can be summarized as follows:

- Natural systems assessment and economic valuation must become an integral part of water resources planning and implementation from the very start of planning.
- The causal sequence: human activity--natural system changes--impact on human health and welfare is an important concept for natural systems analysis.

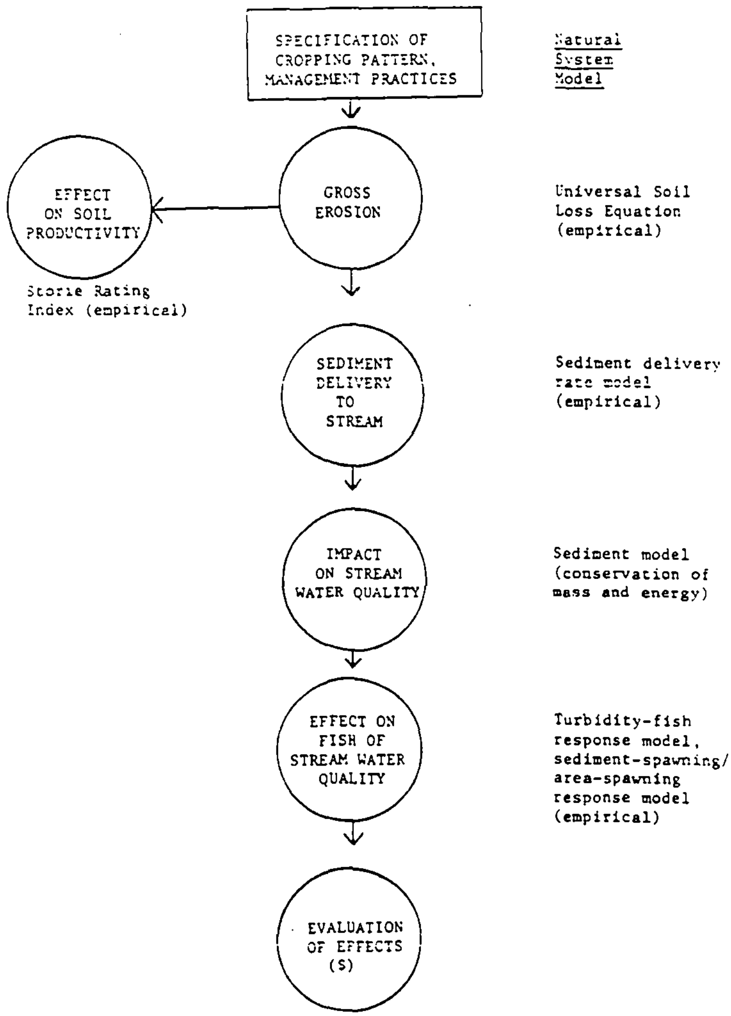

FIGURE 4. Sequence of Analysis for Estimating On-site and Off-site Effects of Agricultural Activities

- The analytical sequence of identification-quantification-valuation is another important concept that is useful as an aid to analysis.
- Models play a key role in quantification of natural systems changes and effects on receptors including humans.
- Economic valuation is an essential element in the entire process. Useful techniques are available for economic valuation of many natural systems effects.
- Where such economic valuation techniques are not available, natural systems effects should be reported in quantitative terms where possible and evaluated in ecological, cultural and aesthetic terms, for consideration by decision-makers.

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Paper number 240 Aspect number 6

ENVIRONMENTAL IMPACT OF ANTI-POLLUTION MEASURES ON AN UNPROTECTED CATCHMENT

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ABSTRACT

In conventional water resources development, catchment areas are usually situated in uninhabited locations where pollution levels are low as a consequence of which raw water quality is of a high order. With the advent of development, particularly in rural catchment areas that are being encroached upon by urbanization, the competing demands for land-use makes this "protected" catchment concept a luxury that man can ill afford. This situation arose in Singapore in 1973 when the Kranji-Pandan Scheme took shape. The water quality in the impounding reservoirs was of a relatively lower order because the runoff was from an unprotected catchment. A vast field pollution survey was carried out and the sources of pollution pin pointed besides which the contributing streams were also monitored. Based on the findings of the survey, massive anti-pollution measures were taken by restructuring the farming sector and improving human waste treatment systems. The runoff and reservoir water quality were monitored throughout and the progressive improvement was remarkable. In this paper, organic levels (represented by BOD and COD) and nutrient contents (represented by PO_4 and Total N) are analysed over a period of time. This study exemplifies the methodology of systematicaly isolating sources of pollution, taking necessary action and proving the efficacy of anti-pollution measures by means of a proper water quality monitoring programme.

Keywords: Water resources development, protected and controlled catchments, field pollution survey, water quality monitoring, organic and nutrient parameters, anti-pollution measures.

INTRODUCTION

In the investigation stage of water resources development projects, one strives to identify sources that warrant minimal treatment. Sources located upstream are, in most countries, less inhabited and consequently, the pollution contributing activities should be of a lower order. In some cases, the activities within these catchments are protected by law (Nature Reserves Act, 1951); such catchments can be identified as "protected" catchments. Before 1969, only such protected catchments were being utilised in the Republic of Singapore. But with rapid industrialization and urban expansion resulting in a spiralling increase in water demands, land scarcity became more pronounced. The luxury of maintaining such protected catchments in rural areas had to be curbed and some activities had to be allowed within catchment areas. The Seletar Scheme was the first of its kind wherein an impoundment in a protected catchment was augmented by pumped inflow from eight unprotected catchments (Arah, 1970). Following the success attained in this Scheme, the Kranji-Pandan Project was embarked upon in 1970 but in this case, the whole catchment area was totally unprotected. There was considerable human, farming and industrial activity within the catchment and the generated waste loads were going to discharge into the reservoir.

THE PROBLEM

Following the closure of the Kranji dam in 1972, monitoring of the reservoir of water quality commenced. The level of pollution within the reservoir (Table 1) was definitely of a higher order when compared with a conventional reservoir lying in a protected catchment (Appan, 1982). Initially the prime concern was the high Cl^- content, but this was countered by continuous pumping out of the impounding water. The main problem, however, was the high organic and nutrient concentrations.

TABLE 1

QUALITY OF RAW WATER

Parameters in mg/L (except pH)	Protected catchment	Unprotected catchment
BOD	3.4	6.7
COD	16.5	86.9
DO	6.1	3.0
SS	7.1	32.4
TDS	59.0	1155.0
Cl^-	8.0	383.0
Total N	8.0	14.0
PO_4 (as P)	0.07	1.22
pH	6.9	7.2

The economic implications as a result of this poorer raw water quality were reflected in the chemical and production costs that were deemed to increase to four and three times as much respectively (Appan, 1977). The type and degree of treatment for this raw water ensured an output quality the standard of which, though of a slightly lower order, is comparable to that from a protected catchment (Pakiam, 1980) and internationally acceptable standards (W.H.O., 1971) as shown in Table 2.

TABLE 2

WATER QUALITY STANDARDS

Parameter in mg/l except pH	Treated water from		W.H.O. International Standards
	Protected catchment	Unprotected catchment	
Colour (Hazens)	< 5	< 5	5
Turbidity (SI Unit)	< 5	< 5	5
pH	7.5	7.5-8.5	7-8.5
Total Hardness	30	130	100
Total solids	50	600	500
Cl^-	14	200	200
Sulphate	8	90	200
Iron	< 0.05	< 0.10	0.10
Copper	-	< 0.005	0.05
PO_4 (as P)	0	< 0.15	-

POLLUTION SURVEY AND RESULTS

Pollution surveys of different types have been conducted in the past, emphasis being placed on the discernible point sources and their impact on a waterbody by monitoring and analysis (Dorfman & Jacoby, 1979). But in this study which embraced a catchment area of 63 sq km, the approach adopted was two fold viz., a field pollution survey and a water monitoring programme.

Field Pollution Survey: A detailed field exercise was carried out in which the catchment was broken down into a number of grids, the smallest area being defined as a microgrid having an area of 202 hectares (500 acres). The pollution load contributers from each of these microgrids were ennumerated and the types of sources, number, degree of treatment (if any) and other information relevant to water pollution was noted. The main waste sources were identified to be from the farming, domestic and industrial sectors (APPAN, 1973). The appraisal of unit pollution loads (Appan & Chin, 1979) clubbed with appropriate field measurements helped in determining the inputs into the reservoir which were budgeted as shown in Table 3 (Appan, 1982).

TABLE 3

INPUT BUDGET FOR BOD AND PO_4

	Waste loads in metric tons/annum					
	Pigs	Humans	Industries	Erosion	Rainfall	Total
BOD	5716 (85%)	252 (4%)	104 (1%)	468 (7%)	190 (3%)	6731 (100%)
PO_4	546 (87%)	75 (12%)	3 (0.5%)	3 (0.5%)	4 (1%)	630 (100%)

The figures clearly identified and quantified the major sources and it was possible to pinpoint the exact locations. With an estimated contribution of not less than 85% of the load coming from the predominantly pig farming sector, the emphasis on the type of source on which action was imminent was well-defined. The human waste loads were attributed to some poor disposal practices still in existence.

Water Quality Monitoring: The area involved in this paper is confined to the Kranji catchment as shown in Figure 1. During the field pollution survey, the streams and their origins were investigated. The sampling points were chosen taking into consideration the accessibility, degree of representation of the subcatchment, facilities for taking flow measurements and the influence of the backwater curve. Based on these criteria, 11 monitoring points were selected and samples collected on a weekly basis, the flow also being measured at the same time. Besides, dissolved oxygen and temperature measurements were done in-situ. The samples were analysed in laboratories for physic-chemical and bacteriological parameters.

For ease of presentation in this paper, only organic (BOD and COD) and nutrient parameters (PO_4 and Total N) will be dealt with. Also only 2 of the 11 streams which are considered to be the most significant and representive will be considered. These streams flow through subcatchments in the western, and eastern areas. The initial values for these parameters in 1974 and the progressive values upto 1981 are presented on an annual weighted mean basis in Figures 2A and 2B. Reservoir sampling was also carried out on a monthly basis and the weighted annual values are presented in Figure 3.

ANTI-POLLUTION MEASURES

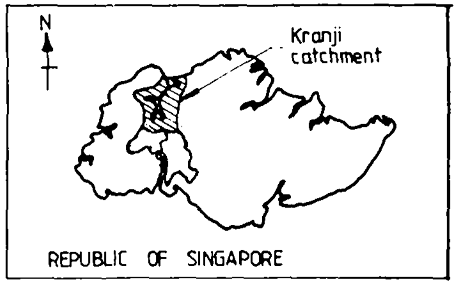
As the main causative agent for the pollution loads was the pigs, initially the idea was to treat the wastes as was done in the Seletar Scheme. But, it was apparent that the voluminious loads encountered were a waste problem of no mean magnitude. Realising the futility and large cost for such treatment, a policy decision was made to resettle the pig farmers to specified areas outside the Kranji catchment where it was envisaged that there was no potential for water resources development. This programme of resettlement commenced in 1973 and the decrease in pigs within the catchment was regularly monitored as was the quality of runoff of the streams. The wasteloads emanating from human sources were reduced by appropriate action being taken to replace the existing poor disposal systems.

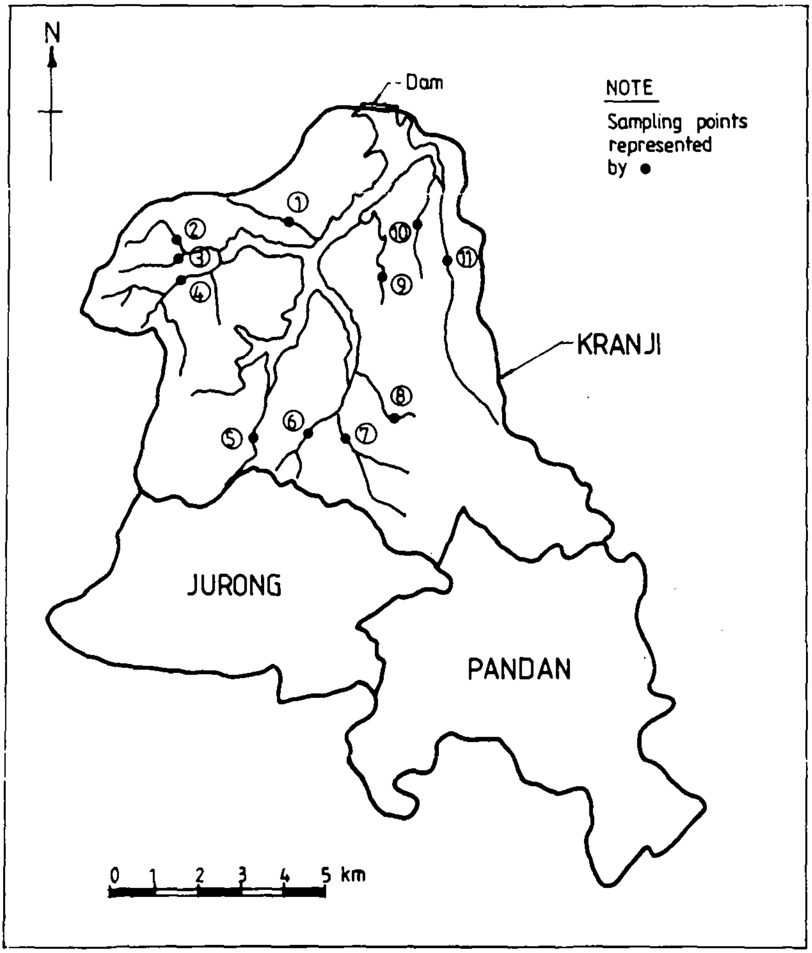
The progressive thinking in the resettlement of pig farmers culminated in the barring of any form of pig farming within this catchment (Cattle Restricted Area Notification, 1977).

Besides, separate effluent standards were set for allowable waste disposal levels for all other form of activity within this catchment. Such areas were classified as "controlled" catchments and a new set of allowable effluent standards was legislated in 1976, the levels being of a more stringent order than those permitted in non-catchment areas as shown in Table 4 (The Trade Effluent Regulations, 1976).

EFFECT OF RESETTLEMENT OF PIGS ON RUNOFF

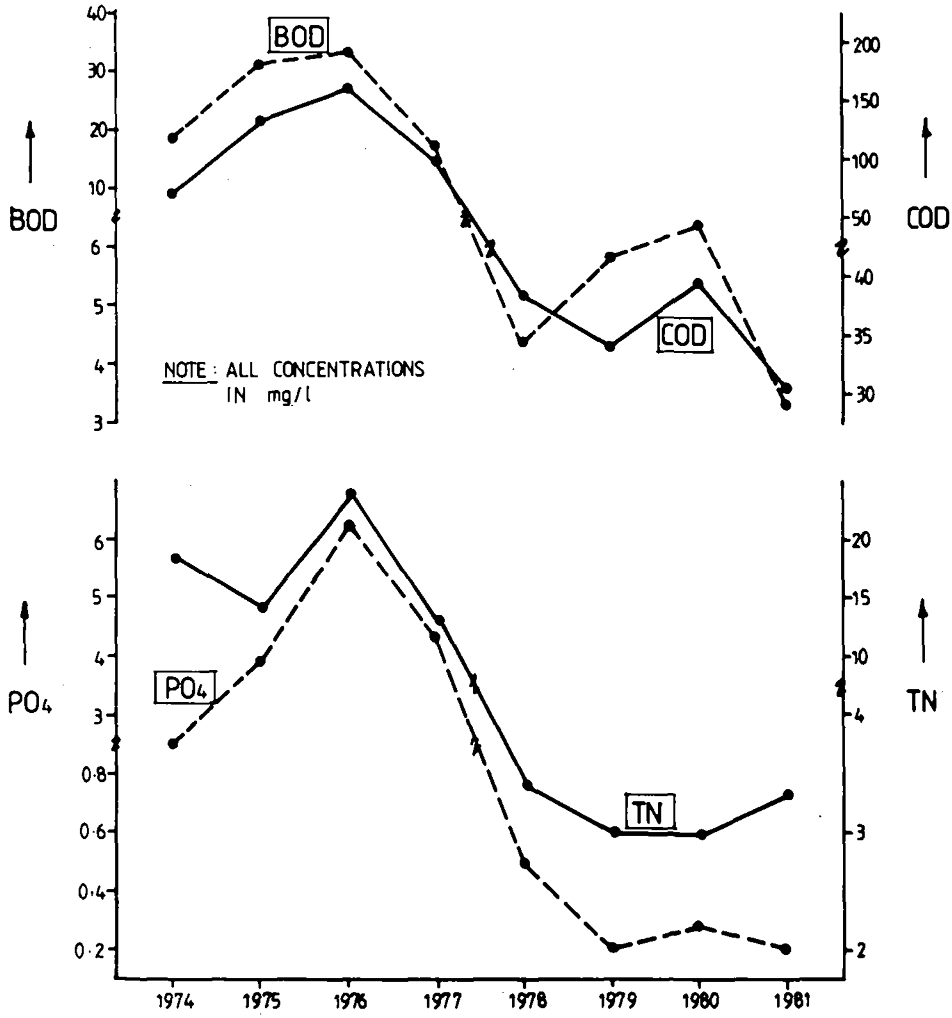
The main aim of the pollution survey and subsequent legislative measures was to enhance runoff and reservoir quality. The progressive decrease in the pig population is shown in Figure 4 for the period 1973 to 1979, during





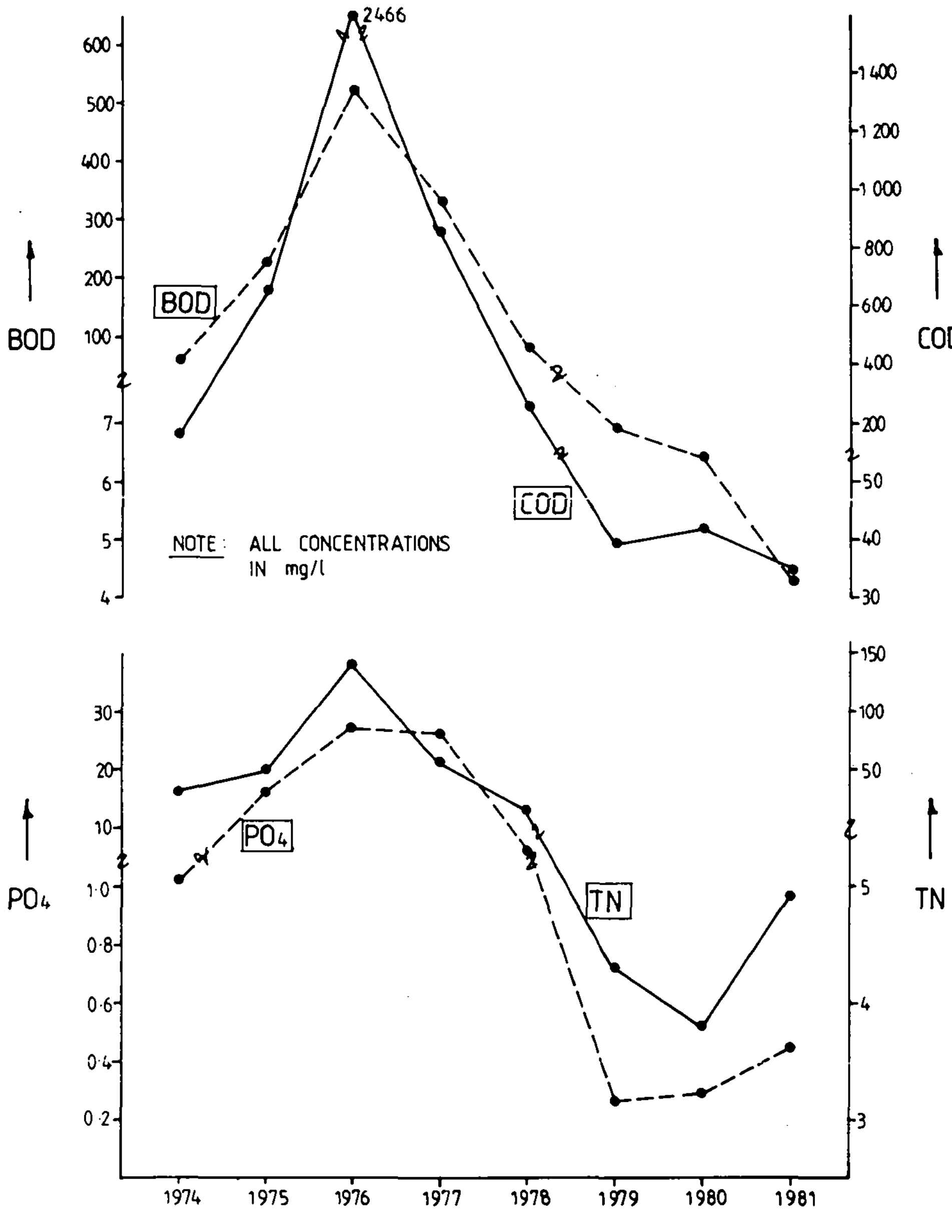
KRANJI CATCHMENT - SAMPLING LOCATIONS

FIGURE 1



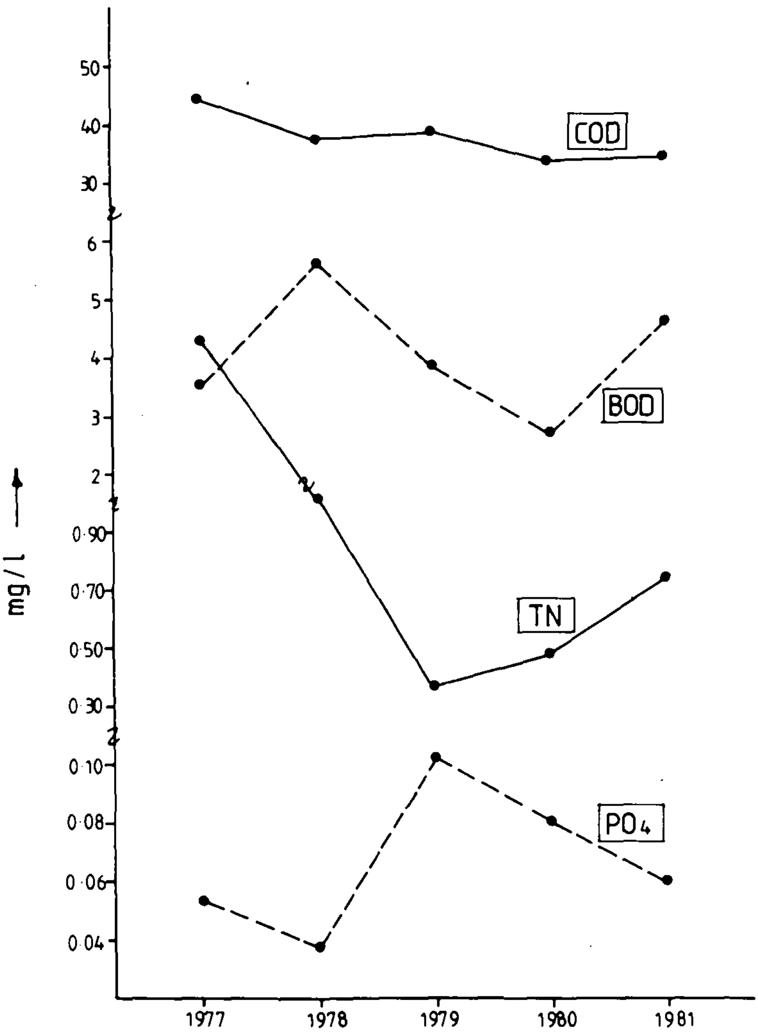
SAMPLING POINT NO. 9

FIGURE 2A



SAMPLING POINT NO. 4

FIGURE 2 B



RESERVOIR QUALITY

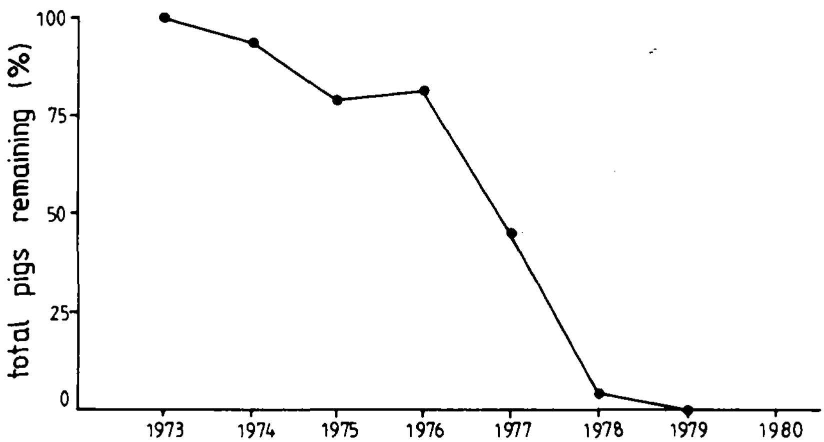
FIGURE 3

TABLE 4

TRADE EFFLUENT REGULATIONS

Parameters in mg/l except pH	Uncontrolled catchment	Controlled catchment
Colour (Hazens)	7	7
pH	6-9	6-9
BOD	50	50
COD	100	100
SS	50	30
TDS	2000	1000
Chloride	600	400
Iron	20	1
Phosphate (as PO_4)	5	2

the period the law took full effect. The progressive values in the water quality in the 2 streams (Figures 2A and 2B), show a remarkable improvement over this period of time. The reservoir quality has also improved considerably (see Figure 3) where the decrease in all four parameters is



PERCENTAGE DECREASE IN PIGS

FIGURE 4

very significant. It is anticipated that over a further period of time, input and reservoir quality will generally improve but the nutrient loads may remain stagnant as the existing benthos, indicates sufficiently high levels to warrant action for reduction in concentration (Appan et al, 1981).

CONCLUSIONS

This exercise exemplifies the concept that environmental activity controls runoff quality and so any action on the environment should have an appreciable impact on runoff and reservoir quality.

Field pollution surveys though very costly and tedious have been instrumental in pinpointing the sources of pollution and, to a large extent, quantifying them. They have helped to formulate policy in the matter of anti-pollution measures and have been instrumental in defining the line of action to be taken.

Consequently, a law has been promulgated for the control of pig farming in specific areas. This line of thinking has culminated in the understanding that the production of indigenous pork is so costly in terms of associated waste management that pig farming should be totally discouraged throughout the Republic (Straits Times, 1984).

An awareness is there that the land within which our water resources lie cannot be fully protected in the traditional sense. Hence, the "controlled" catchment concept has evolved, a set of standards set for effluent discharges within these areas and a law promulgated.

In conclusion, the pollution survey established the causes for the poor runoff and, consequently, poor reservoir quality. The anti-pollution measures taken on the salient pollution-contributing factors based on these findings and the progressive monitoring of the runoff quality have, in effect, established the relationship between cause and effect and should justify, the modus operandi for such projects.

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PLANNING MODEL FOR THE OPERATION OF A MULTIRESERVOIR SYSTEM

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ABSTRACT

This paper presents a methodology to obtain optimal reservoir operation policies for a large-scale reservoir system. The model yields medium-term (one-year ahead) optimal release policies that allow the planning of activities within the current water year, with the possibility of updating preplanned activities to account for uncertain events that affect the state of the system. The solution method is a sequential dynamic decomposition algorithm that keeps computational requirements and dimensionality problems at low levels. The model maximizes the system annual energy generation. The adequate fulfillment of other system functions is guaranteed via constraints on storages and releases. River flows are characterized as a multivariate autoregressive process and are forecasted using maximum likelihood estimators. The model is applied to a large-scale multireservoir system, the northern portion of the California Central Valley Project. The optimal release policies show a potential increase in the system total annual energy output with respect to heuristic schedules currently in use

Keywords: Reservoir operation, multipurpose reservoirs, release policy, energy generation, streamflow forecasting, optimization model, sequential optimization, linear programming.

INTRODUCTION

Previous research on reservoir operation models point out two main difficulties associated with the computation of release schedules for multi-unit systems. First, the decision space is usually vast (dimensionality) and second, the incorporation of the stochastic nature of some elements (mainly inflows) introduces statistical complexities. Effective reservoir operation requires detail (i.e., that release schedules be computed for each reservoir) and validity (i.e., that policies be consistent with the actual realization of uncertain events) of any proposed release schedule. This paper develops an optimization model to compute release policies for large-scale reservoir systems. The model yields policies for every component of the network and keeps these policies up-to-date with the actual realization of streamflows, thus providing detailed and valid operation policies. The remainder of this paper contains a system and problem description, a formulation of the optimization model, and an application to the northern portion of the California Central Valley Project (NCVP). A set of conclusions summarize our findings. The model and application presented in this paper should be of particular interest to managers of multireservoir systems where the reduction of the dimensionality of the reservoir operation problem is critical.

SYSTEM AND PROBLEM DESCRIPTION

The system under analysis is composed of the following reservoirs: (1) Clair Engle, (2) Lewiston, (3) Whiskeytown, (4) Shasta, (5) Keswick, (6) Folsom, (7) Natoma, (8) New Melones, and (9) Tullock. Figure 1 shows a schematic representation of the NCVP system and the points at which accretions/diversions occur. The NCVP is managed jointly by the U.S. Bureau of Reclamation and the California Department of Water Resources. The main purposes of the NCVP are: provision of water for irrigation, municipal and industrial uses, environmental control and enhancement, fish and wildlife requirements, river navigation, water quality control, flood regulation, hydropower, recreation, and control of ocean intrusion and erosion.

The system release policy is subject to physical and technical constraints that arise from the capacity and technological features of the facilities, as well as institutional and environmental regulations. Mariño and Loaiciga (1983) provided a detailed quantitative description of constraints imposed on the operation of the NCVP system that arise from the existence of the multiple functions outlined earlier. Other unique considerations affecting the NCVP system operation are related to the need of extensive fishery operations and salinity control requirements at the Sacramento-San Joaquin Delta. The total power output of the system is delivered mostly to public electric utilities and is used as peaking capacity to satisfy power demands in California. The longest-term operation activities of the NCVP are planned for each water-year. On October 1, the U.S. Bureau of Reclamation estimates future streamflows for the next 12 months. Based on that forecast, a tentative release policy is proposed for the 12-month period. Because actual streamflows deviate from their forecasted values and institutional and/or technical conditions may vary from month to month, streamflow forecasts are updated at the beginning of each month and the release policy is revised for the remaining months of the year. The proposed optimization model of the NCVP system is developed to fit this recurrent monthly scheme for release policies. The updating scheme takes into consideration the most recent streamflow information and the actual system storage evolution.

By far, the stochasticity of flows is the major source for needing to adjust previous policies. The quality of release policies, computed at the beginning of each month and followed strictly for the current month only, depends on the accuracy of the streamflow forecasts. Mariño and Loaiciga (1983) developed a multivariate autoregressive (AR) streamflow forecasting technique that takes into account the cross-correlations between different streamflow stations and permits the introduction of multiple lags in the AR process. They estimated the AR parameters via maximum likelihood and utilized the estimated AR process for recursive streamflow forecasting. Their approach permits statistical testing of the order of the AR process as well as of some of the assumptions on the noise term, and of the time invariance properties of the AR parameters. The application of the AR forecasting technique to the rivers of the NCVP system yielded accurate monthly flow estimates that ranged between ± 10% from actual observed values.

The updating scheme utilized in this study consists of a sequential solution of multistage deterministic problems in which reforecasted flows enter in the formulation of each new problem, and initial storages are set equal to the actual values at every beginning of month. Such updating scheme has been successfully applied in control theory and is usually referred to as the certainty-equivalence controller (CEC) or under some circumstances as open-loop feedback controller (Bertsekas, 1976). Each multistage deterministic problem is decomposed into a sequence of two-stage problems (that are solved one at a time) within the framework of the progressive optimality algorithm (POA) (Howson and Sancho, 1975), leading to low storage and computational requirements. Figure 2 illustrates the solution process by the POA for the first three months during the kth iteration. The first two-stage problem involves months 1 and 2. The beginning and ending storage vectors $\underline{x}_1$ and $\underline{x}_{13}$, respectively, are fixed (the application below discusses the choice of $\underline{x}_{13}$). To solve the first two-stage problem, the current value of $\underline{x}_3^{(k)}$ is held fixed and the optimization is carried out with respect to $\underline{x}_2$ to obtain $\underline{x}_2^*$, which in turn becomes the (k+1)th iterate value of $\underline{x}_2$. Advancing to the second two-stage problem, $\underline{x}_2^{(k+1)} = \underline{x}_2^*$ and $\underline{x}_4^{(k)}$ are held fixed and $\underline{x}_3$ is optimized to yield $\underline{x}_3^*$. A sweep from months 1 to 12 completes the kth iteration. Several iterations are performed until an adequate user-specified criterion is satisfied. We have introduced some modifications in the POA such that it is not necessary to advance in each iteration from months 1 to 12. The POA has been modified in such a way that successive local optimizations are made at some periods in which relatively higher improvements in the objective function are observed, as depicted in Fig. 3, where the modified advancing scheme for the POA is illustrated.

FORMULATION OF THE OPTIMIZATION MODEL

Adopting a one-year planning horizon with monthly decisions, the following variables are defined: $\underline{u}_t$ is an n-dimensional decision vector of releases at the beginning of month t; its components are u_t^i where i refers to the ith reservoir; and $\underline{x}_t$ is an n-dimensional state vector of storages at the beginning of month t; its components are x_t^i. The time index t goes from t = 1 to t = N + 1 = 13, and n is the number of reservoirs in the system.

Development of two-stage problems

The objective of the optimization problem is to maximize the system annual energy production. Other functions like flood control, water supply, etc., are adequately satisfied by appropriate definition of the constraint set on storages and releases. Energy production is the only revenue generating activity of the NCVP. Other functions are specified by contractual and institutional regulations that are expressed mathematically as a set of constraints as shown below.

The development of the optimization model consists of characterizing the mathematical structure of the two-stage problems in the POA (which involve months t-1 and t). The continuity equation for the NCVP system for month t is

$$\underline{x}_{t+1} = \underline{x}_t + \Gamma_1 \underline{u}_t + \underline{w}_t \tag{1}$$

in which Γ_1 describes the topological connections between the reservoirs; and $\underline{w}_t = \underline{y}_t - \underline{R}_t$, where $\underline{y}_t$ and $\underline{R}_t$ are vectors of forecasted streamflows and diversions, respectively (Fig. 1). Net losses (i.e., direct rainfall minus evaporation and seepage) have been neglected due to their small effect in the water balance of the NCVP (Mariño and Loaiciga, 1983). From Eq. 1 follows that the releases $\underline{u}_t$ can be solved for in terms of storages, streamflows, and diversions. Clearly, $\underline{u}_{t-1}$ can also be derived in an analogous manner. Equation 1 is useful in deriving the structure of the objective function in the two-stage problems. Such objective function is given by

$$\text{Maximize } E_t + E_{t-1} \tag{2}$$

in which E_t is the system total energy during month t (in megawatt hours, MWh). E_t can be derived analytically by expressing it as follows:

$$E_t = \xi_t^i \, u_t^i = [a^i + b^i(x_t^i + x_{t+1}^i)]u_t^i \tag{3}$$

in which ξ_t^i is a linear energy rate in MWh/10^3 acre-ft at the ith reservoir power plant; u_t^i is the release from reservoir i during month t; and a^i and b^i are coefficients determined from energy generation records by regression analysis. The sum of x_t^i and x_{t+1}^i in Eq. 3 expresses the fact that average storage is used to compute E_t. We have found the energy generation rates ξ_t^i to provide accurate energy production estimates, within ± 2% of actual values for given average storages and releases. The basis for deriving the ξ_t^i equations are well analyzed performance data of the NCVP power plants. The accuracy of predictions by the ξ_t^i rates shows that for planning studies it is quite adequate to utilize equations like Eq. 3 to estimate power production. In particular, it is not necessary to recourse to complex modeling of turbine performance curves, resulting in a substantial simplification of the analytical structure of the model. For the whole system, the total energy generated during any month t can be expressed as

$$E_t = [\underline{a} + B(\underline{x}_t + \underline{x}_{t+1})]^T \underline{u}_t \tag{4}$$

in which $\underline{a}^T$ contains the constant terms a^i in Eq. 3; B is a diagonal matrix whose (diagonal) terms are the b^i's in Eq. 3; and $\underline{u}_t$ is a 9-dimensional vector of releases. A similar expression can be written for month t-1.

Recalling that the POA maximizes a sequence of two-stage problems, i.e., maximize $E_{t-1} + E_t$ for t = 2, 3,..., 12 subject to a set of constraints, it follows that the two-stage objective function is

$$E_{t-1} + E_t = [\underline{a} + B(\underline{x}_{t-1} + \underline{x}_t]^T \underline{u}_{t-1} + [\underline{a} + B(\underline{x}_t + \underline{x}_{t+1})]^T \underline{u}_t \tag{5}$$

By solving for $\underline{u}_t$ in Eq. 1 and for $\underline{u}_{t-1}$ in a similar expression, and after substituting $\underline{u}_t$ and $\underline{u}_{t-1}$ into Eq. 5, the two-stage objective function becomes

$$\underset{\underline{x}_t}{\text{Maximize}}\ \underline{h}_t^T \underline{x}_t \tag{6}$$

in which

$$\underline{h}_t^T = (\underline{x}_{t-1}^T - \underline{x}_{t+1}^T)[BG - (BG)^T] - (\underline{w}_{t-1}^T + \underline{w}_t^T)\ (BG)^T \tag{7}$$

where $G = \Gamma_1^{-1}$. Notice that $\underline{x}_{t-1}$ and $\underline{x}_{t+1}$ are known and fixed values as discussed earlier when the POA was described. Clearly, the objective function is in terms of the unknown current month storage $\underline{x}_t$. The two-stage problem for month t is completed by appending a set of constraints to Eq. 6. The constraint set for month t consists of:

$$\underline{u}_t \leq \underline{U}_{t,max}, \quad \underline{u}_t \geq \underline{U}_{t,min} \tag{8), (9}$$

$$\underline{c}_t^{kT}\ \underline{u}_t \geq D_t^k,\ \forall\ k \tag{10}$$

$$\underline{x}_t \geq \underline{X}_{t,min}, \quad \underline{x}_t \leq \underline{X}_{t,max} \tag{11), (12}$$

in which $\underline{U}_{t,min}$ and $\underline{U}_{t,max}$ are minimum and maximum releases, respectively; $\underline{c}_t^k$ is a vector representing the linear combination of releases at demand point k; D_t^k is the water demand at point k; and $\underline{X}_{t,min}$ and $\underline{X}_{t,max}$ are minimum and maximum storages, respectively. A set of similar constraints also holds for month t-1. By substituting releases $\underline{u}_t$ and $\underline{u}_{t-1}$ in the constraint set by their respective expressions in terms of storages, streamflows, and diversions (see discussion after Eq. 1), the constraint set of the two-stage problem corresponding to months t-1 and t can be expressed as

$$A\underline{x}_t \leq \underline{b}_t \tag{13}$$

in which

$$A = \begin{bmatrix} G \\ -G \\ -G \\ G \\ \underline{c}_t^{kT} \\ -\underline{c}_t^{kT} \\ -I \\ I \end{bmatrix}, \quad \underline{b}_t = \begin{bmatrix} -\underline{U}_{t,min} + G\underline{x}_{t+1} - G\underline{w}_t \\ -\underline{U}_{t,min} - G\underline{x}_{t+1} + G\underline{w}_{t-1} \\ \underline{U}_{t,max} - G\underline{x}_{t+1} + G\underline{w}_t \\ \underline{U}_{t-1,max} + G\underline{x}_{t+1} + G\underline{w}_{t-1} \\ -D_t^k + \underline{c}_t^{kT}\ G\underline{x}_{t+1} - \underline{c}_t^{kT}\ G\underline{w}_t \\ -D_t^k - \underline{c}_t^{kT}\ G\ \underline{x}_{t-1} - \underline{c}_t^{kT}\ G\ \underline{w}_{t-1} \\ -\underline{X}_{t,min} \\ \underline{X}_{t,max} \end{bmatrix} \tag{14}$$

where I is a 9 x 9 identity matrix. Summarizing, the structure of the two-stage problem is given by

$$\underset{\underline{x}_t}{\text{Maximize}}\ \underline{h}_t^T\ \underline{x}_t \tag{15}$$

subject to

$$A\ \underline{x}_t \leq \underline{b}_t \tag{16}$$

$$\underline{x}_{t-1},\ \underline{x}_{t+1} \text{ fixed and known} \tag{17}$$

Evidently, Eqs. 15-17 describe a linear programming (LP) model, written in terms of the unknown storage $\underline{x}_t$. After the POA is applied to a sequence of two-stage problems, and convergence to an optimal solution is obtained (i.e., $\underline{x}_t^*$, $t = 2,3,\ldots,12$, is available), the release sequence $\underline{u}_t^*$, $t = 1,2,\ldots,12$, can be recovered from the continuity equation which uniquely relates storages and releases.

In the POA, the beginning and ending storage vectors $\underline{x}_1$ and $\underline{x}_{13}$, respectively, are fixed. Vector $\underline{x}_1$ is specified at the beginning of period 1, and vector $\underline{x}_{13}$ (= $\underline{x}_{N+1}$) must be a value ranging from 1/2 to 2/3 of the capacities of the reservoirs. This range has been established as satisfactory from past experience. If desired, the value of $\underline{x}_{N+1}$ can be updated at every end of the month. This study adopted a value of 7/12 of reservoir capacity, which was not updated because it proved satisfactory.

The sequential solution procedure can be summarized as follows: (1) The initial and final states $\underline{x}_I$ and $\underline{x}_{13}$ are fixed. Subindex I can take values 1 through 11, depending on which month the future release policy is being computed. Forecast flows for the remaining (13-I) months, develop (or adjust) an initial feasible state trajectory $\{\underline{x}_t^{(k)}\}$ (see below), and set the iteration counter k equal to 1 and t = I; (2) Set the time index t = t+1 and solve the LP two-stage problem (Eqs. 15-17); (3) Denote the solution of step 2 by $\underline{x}_t^*$. Set $\underline{x}_t^{(k+1)} = \underline{x}_t^*$ and go to step 2. Repeat steps 2 and 3 until a complete iteration sweep is performed (t = I,..., N). This ends the kth iteration; (4) Perform a convergence test, e.g., is

$$\left| \left[(x_t^i)^{(k+1)} - (x_t^i)^{(k)} \right] \Big/ (x_t^i)^{(k)} \right| \leq 0.01$$

for all i and t (t = I,...,12)? If yes, go to step 5. Otherwise, set k = k+1 (provided that a maximum iteration limit is not exceeded), t = I and go to step 2; and (5) Apply the optimal policy for current month I. Set I = I+1 and go to step 1.

APPLICATION AND ANALYSIS OF RESULTS

The optimization model was tested with average inflow conditions (water year 1979-80, total inflow of 13,936 x 10^3 acre-ft). Two different initial policies were developed for 1979-80. This was done to determine if different initial policies result in distinct optimal policies. Initial operation policies for the NCVP were developed by using a trial-and-error procedure that considers some heuristic criteria used by NCVP managers to set up their release policies. In essence, desired reservoir storages at the end of the water year are selected and a feasible (initial) release policy that achieves those targets is chosen.

Optimal state trajectories (i.e., end of month storages) and their corresponding release policies were obtained by applying the POA to the initial policies and both resulted in the same solution. Table 1 summarizes the energy production levels obtained for water year 1979-80 by using the optimization model and actual operation schedules. The ratio of actual energy (E_a) and maximized energy (E_m) varied from 29% at New Melones power plant to 72% at Shasta power plant. Those ratios should be interpreted as an indication of the potential that exists to improve energy generation levels. At New Melones, e.g., legal battles of environmental origin kept the reservoir from being filled completely, and also the power plants were in complete halt during three months, which affected the actual power production adversely. From the results of the model, it can be estimated that for water year 1979-80 there could have been a potential increase of up to 30% in power production in the NCVP. The actual realizable increase is definitely lower.

Benefits of the optimization model can also be measured in terms of increased water deliveries to downstream users. For example, the Delta requires a delivery of 3,850 x 10^3 acre-ft of water per year. Optimal release policies indicated a total annual release of 12,627 x 10^3 acre-ft (for 1979-80), more than three times the required amount. For May-August, when most agricultural activities take place, additional water could be supplied for leaching and crop-growing purposes. The Delta requirements for May-August are about 2,698 x 10^3 acre-ft. For the same period, optimal releases indicated that 4,813 x 10^3 acre-ft were delivered in 1979-80. This suggests the possibility of a conjunctive use of surface water and groundwater reservoirs. Also, with increased deliveries, cultivated areas could be expanded or better leaching of salts might be achieved, resulting in an expanded economic output. Fish spawning, water quality, and navigation would also benefit from increased water deliveries.

CONCLUSIONS

Several conclusions can be drawn from this study. (1) It is possible to increase the annual energy production of the NCVP system as shown for a year of average flow conditions. (2) Delta and agricultural water deliveries can be increased by adopting the optimal release policies. This suggests the possibility of increasing irrigated areas, providing better leaching of agricultural fields, and improving conjunctive management of surface water and groundwater reservoirs. (3) Much of the improvement achieved by the optimal operation policies developed in this study relative to the actual implemented operation schedules is due to: (i) an accurate river inflow forecasting technique; and (ii) an integrative analysis, intrinsic in the optimization model, that allows to represent all the links and constraints that act simultaneously and interactively within the system. Clearly, this integral conceptualization cannot be achieved by a heuristic approach based solely on experience. (4) The improved performance reported by the use of the optimization model should be viewed as an upper bound to the possible gains that could be derived from the use of a mathematical model. Also, the more knowledgeable the system managers become with the reservoir optimization model, the closer the performance of the system will be to the upper bounds obtained under the conditions assumed by the model. Clearly, the use of a mathematical model and the better understanding that emanates from its use should result in a feedback to the model, with its probable reformulation and modification that would bring closer the unpleasant difficulties of any real-world system and the sophistications inherent to any mathematical model.

TABLE 1. Actual (E_a) and Maximized (E_m) Energy Production (in 10^3 MWh) Corresponding to Water Year 1979-80.

Month	Trinity Power Plant at Clair Engle		Judge Francis Carr Power Plant		Spring Creek Power Plant		Shasta Power Plant		Keswick Power Plant		Folsom Power Plant		Nimbus Power Plant at Lake Natoma		New Melones Power Plant	
	Actual	Max.	Actual	Max.	Actual	Max.	Actual	Max.	Actual	Max.	Actual	Max.	Actual	Max.	Actual	Max.
Oct	30.4	73.9	36.8	112.4	42.3	109.0	76.5	112.1	19.0	46.6	37.7	53.5	4.6	6.4	*	71.6
Nov	9.6	71.2	4.8	112.4	19.5	109.5	89.6	126.2	20.7	50.3	41.3	77.9	4.7	10.0	*	70.4
Dec	17.1	61.4	15.4	98.5	19.2	98.5	111.1	251.3	23.2	81.6	37.2	71.3	4.7	10.0	*	68.1
Jan	6.2	29.7	*	41.9	23.9	77.2	237.4	266.9	47.6	81.6	107.0	118.4	8.7	10.0	19.6	81.9
Feb	17.7	31.1	3.4	41.9	55.3	41.8	209.9	310.7	39.2	81.6	84.3	139.9	6.7	10.0	23.6	92.1
Mar	73.4	32.2	78.2	41.9	99.5	42.8	228.3	323.7	49.9	81.6	130.0	147.2	10.6	10.0	47.8	90.9
Apr	44.9	32.9	53.5	41.9	54.7	53.2	112.9	126.1	29.0	37.3	99.8	126.3	9.7	7.7	55.8	90.4
May	21.2	33.7	21.2	41.9	18.6	49.3	164.4	128.8	33.7	36.6	82.0	131.0	9.1	8.2	39.5	85.8
Jun	51.5	75.3	55.2	101.2	59.6	99.4	212.9	129.5	47.3	45.0	66.6	76.4	7.6	7.8	23.3	84.4
Jul	54.3	39.4	57.2	44.7	57.2	45.7	237.9	307.5	52.2	77.0	84.4	63.4	9.7	6.5	38.5	80.9
Aug	75.7	37.6	87.2	42.9	85.9	43.5	165.7	310.2	43.6	81.6	35.6	93.5	4.3	10.0	22.1	74.9
Sep	71.1	36.5	84.7	44.2	88.6	43.5	86.8	289.0	29.3	81.6	40.5	86.8	4.6	10.0	9.0	64.8
Total	473.1	817.5	497.6	765.9	624.3	813.7	1933.4	2682.2	434.7	782.8	846.4	1185.5	85.0	106.5	279.2	956.5
E_a/E_m	0.58		0.65		0.77		0.72		0.56		0.71		0.80		0.29	

* Power plant not in operation.

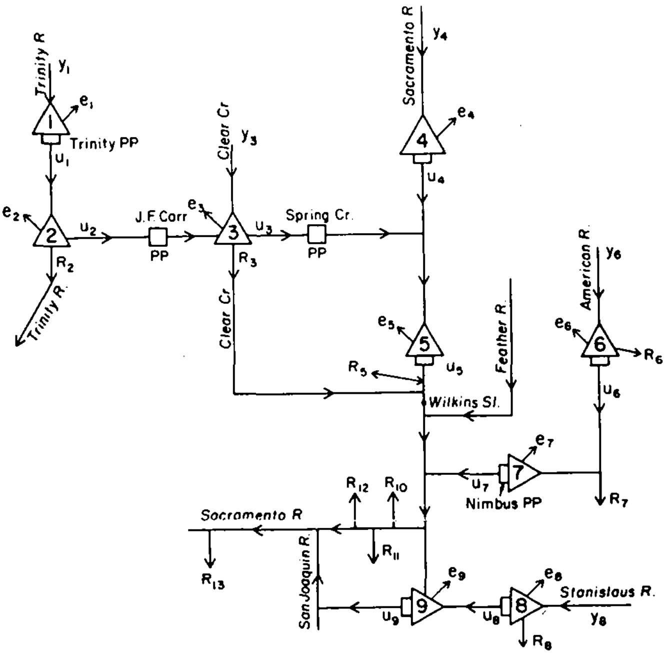

Fig. 1. Schematic representation of the NCVP showing diversions (R), net losses (e), releases (u), and streamflows (y).

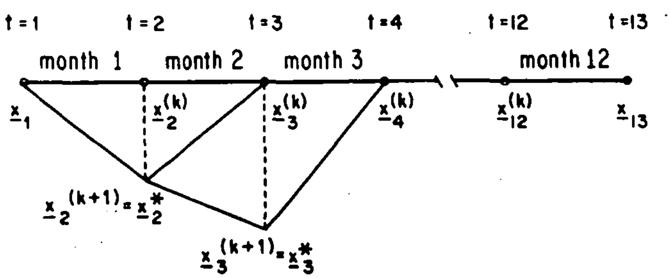

Fig. 2. Sequential optimization scheme.

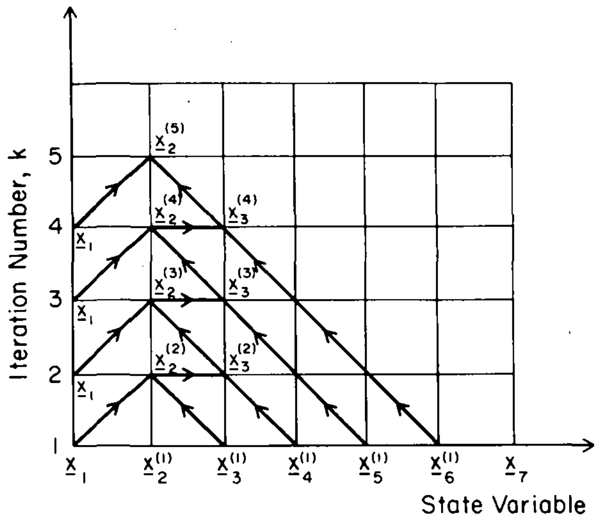

Fig. 3. Modified POA (To achieve state $\underline{x}_2^{(5)}$: (i) $\underline{x}_1$ and $\underline{x}_3^{(1)}$ yield $\underline{x}_2^{(2)}$; (ii) $\underline{x}_2^{(2)}$ and $\underline{x}_4^{(1)}$ yield $\underline{x}_3^{(2)}$; (iii) $\underline{x}_1$ and $\underline{x}_3^{(2)}$ yield $\underline{x}_2^{(3)}$; (iv) $\underline{x}_2^{(3)}$ and $\underline{x}_5^{(1)}$ yield $\underline{x}_3^{(3)}$; (v) $\underline{x}_1$ and $\underline{x}_3^{(3)}$ yield $\underline{x}_2^{(4)}$; (vi) $\underline{x}_2^{(4)}$ and $\underline{x}_6^{(1)}$ yield $\underline{x}_3^{(4)}$; and (vii) $\underline{x}_1$ and $\underline{x}_3^{(4)}$ yield $\underline{x}_2^{(5)}$). *This scheme should be used when significant* improvements (e.g., 5% improvement in the objective function with respect to the previous iteration) arise from the two-stage problem involving periods 1 and 2).

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GESTION INTEGREE DU COMPLEXE DES BARRAGES DE L'EAU D'HEURE ETABLISSEMENT DU GRAPHIQUE DES CAPACITES OPTIMALES SUR BASE DE LA PERIODE 1969-1980

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RESUME

Le complexe des barrages de l'Eau d'Heure, mis en service en 1978, a comme principal objectif de soutenir le débit d'étiage de la Sambre.

En outre ce complexe doit remplir deux autres objectifs : la production d'électricité et le développement d'activités touristiques, objectifs qui doivent se conjuguer le mieux possible avec le premier.

En se basant sur la méthode consistant à simuler le fonctionnement du système pendant une période la plus longue possible (1969-1980) tout en y intégrant des mesures hydrologiques et météorologiques les plus récentes possible, le Service des Barrages a déterminé l'utilisation optimale de ce complexe des barrages.

Dans un premier temps, est déterminée la courbe enveloppe des capacités utiles nécessaires pour assurer un soutien d'étiage de la Sambre au débit minimum garanti de 4,5 m3/sec. (voir annexe n° 1).
Il en résulte la constatation que ce débit est le minimum que les barrages de l'Eau d'Heure peuvent garantir en cas de retour d'une période semblable à celle de 1969-1980.

Dans un second temps, en faisant la distinction entre la contenance totale des deux lacs (Eau d'Heure + Plate Taille) et leur contenance utile c'est-à-dire le volume qui peut être effectivement utilisé pour le soutien d'étiage, est déterminée un second graphique celui des capacités optimales (voir annexe n° 1).

Il en résulte qu'une série de consignes peut être prescrite à l'usage du responsable de la gestion de ce complexe, consignes qui tiennent naturellement compte des trois objectifs assignés à ce complexe.

Gestion intégrée du Complexe des Barrages de l'Eau d'Heure. Etablissement du graphique des capacités optimales sur base de la période 1969-1980.

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INTRODUCTION.

Le Complexe des barrages de l'Eau d'Heure, le plus important de Belgique, est situé dans la région de l'Entre-Sambre-et-Meuse à l'intérieur d'un quadrilatère formé par les villes de Philippeville, Couvin, Chimay et Beaumont, à une trentaine de kilomètres de Charleroi.
Il a été mis en service dans le courant de l'année 1978.

Les objectifs assignés à ce complexe résident dans :

- le soutien du débit d'étiage de la Sambre et indirectement de celui de la Meuse avec accessoirement l'écrêtement des crues de l'Eau d'Heure;
- la production d'électricité et
- le développement d'activités touristiques.

La description succincte des ouvrages constituant ce complexe ainsi que la définition des objectifs sont données ci-dessous.

La méthode adoptée pour déterminer l'utilisation optimale de ces ouvrages a consisté à simuler le fonctionnement du système pendant une période la plus longue possible et, compte tenu du perfectionnement progressif des mesures hydrologiques et météorologiques, les plus récentes possible.

Cette méthode revient à supposer que les ouvrages que l'on étudie étaient déjà construits et exploités à l'époque considérée, et à examiner comment ils se seraient comportés alors, en fonction des conditions météorologiques et autres qui ont réellement existés et des consignes d'exploitation que l'on souhaite leur imposer. On peut "injecter" dans les calculs toute hypothèse concernant le développement industriel futur, les rejets, les prélèvements, ... dans le cours d'eau à assainir (en l'occurence la Sambre) ou concernant les autres objectifs des réserves que l'on étudie (production d'énergie électrique, d'eau domestique, tourisme, ...).

En ce qui concerne le Service des Barrages, il s'est, depuis la mi-78, intéressé principalement à la période démarrant en 1969, pour la triple raison que :

- depuis cette date le réseau de stations limnigraphiques couvre pratiquement tous les cours d'eau concernés; et ceci que l'on considère le complexe des barrages de l'Eau d'Heure isolément, ou en liaison hypothétique avec des captages (Ry d'Yves, Eau Blanche, Brouffe, Meuse, ...) ou des réserves (Eau Noire) à réaliser ultérieurement;

- cette période comporte une séquence fort sèche, démarrant vers 1973, culminant avec l'année exceptionnelle que fut 1976 et se terminant en fin 1979;
- elle se situe à la fin d'une décennie fort humide, de telle sorte qu'elle peut être "détachée du passé", et cela quelles qu'aient été les sollicitations des réserves supposées existantes pour pallier des sécheresses antérieures.

La présente étude a pour but de fixer, pour les premières années d'exploitation réelle et en fonction des objectifs actuels et de leur hiérarchie, les consignes aux exploitants hydrauliciens. Il est à noter que les exploitants électriciens ont leurs propres consignes, qui peuvent éventuellement se révéler contradictoires dans certaines situations.
Il appartient aux deux Administrations de se concerter dans ces cas pour fixer les priorités.

DEFINITION DES OBJECTIFS, DES MOYENS ET DES HYPOTHESES.

Les objectifs.

Ils sont multiples et sont classés ici par ordre croissant d'importance du point de vue de l'hydraulicien, afin de développer in fine, la fonction principale des barrages de l'Eau d'Heure, qui est de fournir de l'eau à la Sambre en période d'étiage.

Les activités touristiques :

Pour satisfaire à ce critère, il faut et il suffit de :

- ne jamais prélever d'eau dans les lacs des prébarrages;
- éviter de descendre les plans d'eau des lacs principaux en-dessous des cotes indiquées dans le tableau reprenant les caractéristiques hydrauliques de la Plate Taille et de l'Eau d'Heure.

L'écrêtement des crues de l'Eau d'Heure;

Pour satisfaire à cette fonction, dont la périodicité est aléatoire, la seule stratégie est de maintenir un volume libre aux époques où l'étude montre qu'il n'est statistiquement pas nécessaire de disposer de la totalité de la réserve pour fournir de l'eau à la Sambre.
A contrario, il existe donc des époques où la poursuite de l'objectif prioritaire rend impossible de conserver une réserve d'empottement pour satisfaire l'écrêtement des crues de l'Eau d'Heure.

La production électrique :

pour satisfaire à cet objectif deux limites à l'utilisation des réserves d'eau s'imposent :

- une limite supérieure : les deux lacs principaux ne peuvent être remplis simultanément; un "creux" doit subsister pour permettre les transferts d'eau entre lacs, qui constituent la clef du fonctionnement de la centrale électrique;

- une limite inférieure : en-dessous des cotes indiquées ci-dessous au tableau reprenant les caractéristiques hydrauliques des deux lacs, la centrale électrique ne peut fonctionner, par désamorçage des pompes ou par manque de charge.

La fourniture d'eau à la Sambre.

Il s'agit, en fait, tant historiquement que hiérarchiquement, de la fonction première du complexe des barrages de l'Eau d'Heure.
Pour mémoire, l'Eau d'Heure, affluent de rive droite, se jette dans la Sambre à Marchienne-au-Pont, pratiquement à l'entrée du "parcours industriel" de la rivière. Un peu en aval, le canal de Charleroi à Bruxelles prélève son eau dans la Sambre.
Dans toute la traversée de l'agglomération carolorégienne, une importante industrie est établie le long de la rivière avec comme inconvénients, pour la qualité et la quantité des eaux : prélèvements, réchauffements et pollutions diverses.
Plus en aval existe encore le bassin industriel de la Basse Sambre.
La rivière se jette dans la Meuse à Namur. Depuis 1958, c'est à l'écluse de Namur que les débits de la Sambre sont mesurés. Depuis quelques années, le Service d'Etudes Hydrologiques du M.T.P. cherche à améliorer la mesure du débit; divers essais ont eu lieu ou sont en cours en des points divers du parcours Charleroi-Namur.

Le principe qui a prévalu à la construction des barrages est qu'il fallait assurer à la Sambre à la sortie du bassin industriel carolorégien, un débit qui ne descende jamais en-dessous de 5 m^3/s. Sur base des données et des hypothèses de l'époque, les ouvrages ont été dimensionnés ainsi qu'il est indiqué au § ci-après relatif aux "moyens".

Maintenant que les barrages sont construits, le problème peut être posé en sens inverse : quel est le débit minimum qui aurait pu être garanti à la Sambre pendant la période 1969-1980 si les barrages avaient existé avec toutes leurs caractéristiques actuelles ?

Les moyens (description des ouvrages).

Le complexe des barrages de l'Eau d'Heure se compose de cinq barrages, retenant cinq lacs : la Plate Taille, l'Eau d'Heure et trois prébarrages.
Les prébarrages sont destinés à demeurer constamment remplis. Leurs apports se déversent automatiquement dans le lac de l'Eau d'Heure; ils sont aussi inclus automatiquement dans le calcul des apports de l'Eau d'Heure; leurs pertes par évaporation et infiltration, quoique faibles, sont prises en compte dans les calculs. Pour le reste, ils n'interviennent pas dans les manutentions d'eau, ni dans la production électrique. Il n'en sera plus question dans la suite.

La réserve de la Plate Taille.

C'est le principal stockage pour l'eau destinée au soutien d'étiage de la Sambre. C'est en même temps le réservoir supérieur du système de centrale électrique à récupération. La Plate Taille est destinée, en temps normal, à pouvoir fluctuer entre les capacités suivantes : 15,6 hm3 et 67,8 hm3, correspondant aux cotes extrêmes 230 m et 250 m DNG.

En-dessous de la cote 230, la charge sur les turbines devient insuffisante et en outre les installations de mise à eau des bateaux ne permettent plus les activités touristiques principales (voile).
Au-dessus de la cote 250, le lac déborde par les déversoirs.

Le barrage de la Plate Taille est construit sur un affluent minuscule de l'Eau d'Heure, dont les apports propres sont quasi-insignifiants. En première approximation, l'on peut donc considérer que tous les mouvements d'eau de ou vers la Plate Taille transitent par le lac de l'Eau d'Heure, via les conduites forcées de 4,5 m Ø ou via les conduites de premier remplissage de 2,2 m Ø.

La réserve de l'Eau d'Heure.

Elle peut fluctuer entre les capacités extrêmes suivantes : 7,6 hm3 et 17,15 hm3, correspondant aux cotes extrêmes 202 m et 208,40 m DNG. En-dessous de la cote 202, la centrale électrique ne peut fonctionner et les activités touristiques principales (motonautisme, ski nautique) ne peuvent être pratiquées par manque de tirant d'eau. Au-dessus de la cote 208,40, le lac déborde par les déversoirs.
En dehors de son rôle dans la production d'électricité, dont il constitue le réservoir inférieur, la fonction de ce lac est cependant essentielle puisque c'est par lui que transitent toutes les entrées et les sorties d'eau du complexe et qu'il sert notamment de "volant" lorsque par exemple les cours d'eau présentent une crue qu'il serait souhaitable d'empoter dans la Plate Taille, alors que simultanément, la centrale électrique est en régime de turbinage.

La réserve totale (capacité utile).

Le tableau ci-dessous résume les caractéristiques hydrauliques des deux ouvrages principaux :

	Plate Taille		Eau d'Heure	
	Cote m	Capacité hm3	Cote m	Capacité hm3
Maximum exceptionnel	-	-	208,40	17,15
Maximum normal :	250	67,8	207,00	14,75
Minimum normal pour le fonctionnement électrique et l'exploitation touristique :	230	15,6	202,00	7,6
Minimum absolu :	213	0,8	194,00	1,2
Volume réservé au fonctionnement électrique :	-	6,5	-	6,5

Le second tableau ci-dessous résume le calcul des capacités utiles disponibles (c'est-à-dire : volume pour soutenir l'étiage) dans trois hypothèses différentes :

HYPOTHESES	CAPACITES UTILES DISPONIBLES EN Hm3	
Eau d'Heure rempli niveau max. normal	P.T. : 67,8 - 15,6 - 6,5 = 45,70 E.d'H. : 14,75 - 7,6 - 6,5 = 0,65	46,35
Eau d'Heure rempli niveau max.exceptionnel	P.T. : 45,7 E.d'H. : 17,15 - 7,6 - 6,5 = 3,05	48,75
Abandon production électrique activités touristiques (période très sèche)	P.T. + E.d'H. 48,75 P.T. : 15,6 - 0,8 = 14,8 E.d'H. : 7,6 - 1,2 = 6,4	69,95

Pour ne plus avoir à tenir compte du volume de 6,5 hm3 qui se déplace d'un lac à l'autre et qui n'intervient pas dans la fonction de soutien d'étiage, on peut figer le système dans la situation existant normalement en fin de journée, entre la fin du turbinage diurne et le début du pompage nocturne : à ce moment, le creux se trouve dans le lac supérieur.

A partir de ces considérations et sur base des caractéristiques hydrauliques figurant dans les deux tableaux ci-dessus, on peut déduire la capacité maximale de la Plate Taille, celle minimale de l'Eau d'Heure, les capacités utiles de la Plate Taille et de l'Eau d'Heure et enfin les limites maximum et minimum de la contenance totale des deux lacs.

Capacité maximum de la Plate Taille :	67,8 - 6,5	= 61,3 hm3	
Capacité maximum de l'Eau d'Heure :	7,6 + 6,5	= 14,1 hm3	
Capacité utile de la Plate Taille :	61,3 (max.)	- 15,6 (min.)	= 45,7 hm3
Capacité utile de l'Eau d'Heure :	17,15(max.)	- 14,1 (min.)	= 3,05 hm3
Contenance :	78,45 hm3 max.	29,7 hm3 min.	48,75 hm3 utile max. théorique

Il est à noter que, dans la gestion réelle du complexe, rien n'oblige les électriciens à utiliser chaque jour la totalité des 6,5 hm3 qui leur sont réservés; et que par ailleurs, dans des situations intermédiaires, où aucun impératif hydrologique ne s'y oppose, il n'y a pas de raison de les empêcher de turbiner et pomper tout le disponible de l'Eau d'Heure, soit :

$$17{,}15 - 7{,}6 = 9{,}55 \text{ hm3}$$

Il s'agit là cependant d'éléments qui ne relèvent que des électriciens, et dont nous ne pouvons tenir compte dans nos calculs

Les hypothèses.

débits des rivières.

* Sambre : les débits quotidiens à l'écluse de Namur résultent de :
  - mesures communiquées par le Service de la Sambre, pour la période 1969 à fin 1976.
  - mesures communiquées par l'Office de la Navigation, à partir d'avril 1977.
  - en l'absence de mesures pour le premier trimestre 1977, ces trois mois ont été assimilés à la période hivernale la plus semblable du point de vue de la pluviosité dans le bassin de la Sambre, à savoir : décembre 1973, janvier et février 1974.

* Eau d'Heure et affluents : les débits sont mesurés à hauteur de la gare de Walcourt, à l'aval du confluent avec le Ry d'Yves, et les moyennes quotidiennes sont publiées dans " l'Annuaire Hydrologique " de chaque année.

- pour la période 1969 à février 1977, les débits à hauteur du barrage de l'Eau d'Heure ont été estimés par la formule :

$$Q \frac{E.H.}{barrage} = 0,422 \quad Q \frac{E.H.}{Walcourt} \quad x\ 1,15$$

Le coefficient 0,422 représente le rapport des bassins versants (79 Km2 au barrage, contre 187 Km2 à Walcourt).

Le coefficient multiplicateur 1,15 provient de l'hypothèse admise que les débits spécifiques du bassin limité au barrage sont en moyenne 15 % supérieurs à ceux de l'ensemble du bassin couvert par la station limnigraphique.

- à partir de mars 1977, vu la fermeture des vannes de l'Eau d'Heure, les débits mesurés à l'aval du barrage sont influencés par celui-ci; les valeurs mensuelles du débit sont obtenues en ajoutant au débit restitué à la rivière (mesuré sur chantiers par le Service des Barrages) la différence d'empotement entre la fin et le début du mois.

évaporation et infiltration.

* infiltrations : l'on admet que les infiltrations annuelles représentent globalement 1 % du volume emmagasiné dans l'ensemble des cinq lacs; dans les bilans mensuels les infiltrations sont donc prises chaque mois égales au 1/1200ème du volume contenu à la fin du mois précédent. Lorsque tous les lacs sont remplis, l'infiltration estimée est ainsi de 0,8 hm3 par an, soit près de 70.000 m3 par mois ou encore 25 litres par seconde.

* évaporations : l'évaporation mensuelle mesurée à Cerfontaine par l'I.R.M. est publiée dans les "Annuaires Hydrologiques de Belgique". Le volume évaporé chaque mois par l'ensemble des lacs est estimé en multipliant la hauteur de la lame évaporée par la superficie des lacs à la fin du mois précédent. Pour les mois où il n'y a pas de mesures publiées pour Cerfontaine, on a considéré l'évaporation à la Cuisine. Les hauteurs annuelles évaporées, comprises entre 340 et 478,5 mm, conduisent à pouvoir perdre, si tous les lacs sont remplis jusqu'à 3 hm3 par an.

A partir d'avril 1977, on a admis pour simplifier, que les pertes du complexe en exploitation fictive étaient égales aux pertes réelles des lacs en cours de remplissage. Comme ces dernières sont automatiquement incluses dans le bilan ayant servi à estimer les débits entrants, l'on inscrit zéro dans la colonne des pertes.

EXPLOITATION FICTIVE DU COMPLEXE POUR LA PERIODE 1969-1980.

Variation des capacités et des niveaux.

Les calculs effectués par le Service des Barrages dans les études hydrologiques entreprises depuis 1978, conduisent à des bilans mensuels des réserves. La période de 1 mois est en effet, pour les calculs effectués sans l'aide de l'ordinateur, celle qui nous a semblé réaliser le meilleur compromis entre la durée du calcul et la précision du résultat.

Ces calculs sont cependant basés sur les valeurs quotidiennes du débit manquant dans la Sambre et du débit entrant dans l'Eau d'Heure.

Le principe en est simple : les réserves à la fin d'un mois sont égales aux réserves à la fin du mois précédent, augmentées des apports, et diminuées des prélèvements et pertes.

* les seuls apports sont actuellement les débits des divers adducteurs naturels des lacs. Comme ces débits sont estimés pour la totalité du bassin versant, à hauteur du point le plus en aval du complexe, il ne semble pas opportun de considérer séparément les apports directs de la pluie dans les lacs.

* les prélèvements et pertes se composent actuellement de :

  - débits lâchés, soit pour soutenir l'étiage de la Sambre; soit
    - lorsque les réserves sont à leur niveau optimum - pour évacuer les excédents; soit - en période de reconstitution des réserves - pour ne pas mettre à sec le cours aval de la rivière; pour ce faire, l'on restitue en permanence :

      - 0,2 m^3/s en été (mai à octobre)
      - 0,1 m^3/s en hiver (novembre à avril)

  - pertes par évaporation et infiltrations, telles que décrites ci-dessus

Calcul des capacités nécessaires

Il s'agit ici d'un calcul mené en remontant le temps, de la manière suivante : on part de chacun des points bas d'un graphique d'exploitation fictive et on admet que la capacité utile du réservoir peut être totalement épuisée à cette date; on calcule, pour le dernier jour du mois précédent, le volume minimum nécessaire pour atteindre ce résultat, compte tenu des entrées et sorties du mois considéré; on remonte ainsi dans le temps, mois par mois, en calculant chaque fois le volume nécessaire pour que le volume déterminé à l'étape précédente soit atteint. La courbe ainsi obtenue tracée de droite à gauche, présente un ou plusieurs maxima, dont un maximum maximorum, qui est la capacité utile nécessaire du complexe pour franchir la sécheresse considérée; elle redescend ensuite jusqu'à recouper l'axe des abscisses. Ce point représente une date à laquelle la réserve pouvait être entièrement épuisée sans que cela empêche le barrage de remplir son rôle pendant la sécheresse considérée, c'est-à-dire que, pour autant que le réservoir ait une capacité utile supérieure ou égale à la capacité nécessaire, toute sécheresse antérieure à cette date est indépendante de la sécheresse considérée.

Courbes-enveloppes des capacités nécessaires.

En superposant les courbes de capacité utile dessinées pour chacune des années envisagées dans l'exploitation fictive, on peut tracer (cfr.annexe) une courbe-enveloppe. Celle-ci indique , pour chaque époque de l'année, le niveau jusqu'auquel on peut vider volontairement la retenue (par exemple pour permettre l'empotement de crues éventuelles) sans hypothéquer l'objectif considéré comme primordial : assurer le soutien d'étiage choisi en cas de retour d'une période semblable à la période de référence.

Simulation d'un soutien d'étiage de la Sambre au débit minimum garanti de 4,5 m3/s.

Pour ne pas allonger inutilement cette note, les graphiques figurant pour cette simulation les entrées et sorties d'eau, les variations des plans d'eau et les volumes emmagasinés dans les deux lacs principaux ne sont pas donnés. Seule la courbe enveloppe des capacités utiles nécessaires est reproduite à l'annexe.

A l'examen de cette annexe, on constate que la capacité utile nécessaire est dans ce cas de 51,855 hm3, soit supérieure de 3.105 hm3 à la capacité utile du complexe, telle que définie ci-dessus. Ceci se marque par une réduction du volume turbinable à certaines époques, de l'ordre de 16 % en fin novembre 1976 et en fin décembre 1976 et de l'ordre de 36 % en fin octobre 1979 et fin novembre 1979.

Conclusion :

Pour autant que les diverses hypothèses et mesures rappelées ci-dessus sont valables, il apparaît qu'il manque dans le complexe de l'Eau d'Heure un volume d'environ 3 hm3 de capacité disponible pour assurer un soutien d'étiage de la Sambre au débit minimum garanti de 4,5 m3/s. Ce manque représente 5 % de la capacité utile. Compte tenu de la précision des mesures hydrauliques, on admet que :

> LES BARRAGES DE L'EAU D'HEURE SONT A MEME DE GARANTIR A LA SAMBRE UN DEBIT MINIMUM DE 4,5 m3/s EN CAS DE RETOUR D'UNE PERIODE SEMBLABLE A CELLE DE 1969 - 1980.

CONCLUSION GENERALE

Au § relatif à la "réserve totale", on a fait la distinction entre la "contenance totale" des deux lacs principaux et leur "contenance utile" celle-ci étant le volume qui peut être effectivement utilisé pour le soutien d'étiage.

Cette contenance totale des deux lacs varie entre un minimum de 29,7 hm3 et un maximum de 78,45 hm3; la différence = 48,75 hm3 étant la contenance utile maximale théorique.

Par ailleurs, on a vu à l'annexe quelle capacité utile doit être idéalement maintenue à chaque époque de l'année dans la réserve pour permettre de réaliser l'objectif primordial (soutien d'étiage à 4,5 m3/s.), tout en baissant les niveaux pour l' empotement de crues lorsque cela est possible. Durant quatre mois de l'année - avril à juillet - cette capacité dépasse légèrement la capacité maximale théorique.

Si l'on assimile le "zéro" de la capacité utile à la contenance totale minimale que l'on s'est imposée : 29,7 hm3, les plus grandes valeurs de contenance utile calculées (49,556 à 51,855 hm3) sont supérieures à la contenance totale maximale que l'on s'est imposée :

$$29{,}7 + 49{,}556 = 79{,}256 > 78{,}45 \text{ hm3}$$

Cela signifie qu'il faut tronquer les plus grandes valeurs en les arrasant à la valeur 78,45 hm3.
Le graphique des capacités optimales dans cette solution est représenté à l'annexe

Il appert de ce graphique que le "creux" pour l' empotement des crues paraît suffisant pendant la majeure partie de l'hiver.

Il en résulte les consignes suivantes pour l'exploitation des lacs au point de vue des hydrauliciens :

- la priorité est donnée au soutien d'étiage de la Sambre, de façon à garantir en tout temps un débit supérieur ou égal à 4,5 m3/s.

- pour autant que cela ne contrarie pas cet objectif prioritaire, l'on essaie de moduler les lâchers de façon à obtenir à chaque époque les volumes indiqués à l'annexe pour le total des deux lacs principaux.

Ces consignes sont nuancées et complétées par les considérations suivantes :

- d'avril à juillet, la consigne à remplir les deux lacs principaux à leur capacité max., tout en conservant un creux de 6,5 hm3 pour le fonctionnement de la centrale hydro-électrique; cependant, si les exploitants électriciens ne souhaitent pas utiliser la centrale à sa pleine capacité, le Service des Barrages a intérêt à remplir davantage encore les lacs en juin et en juillet, époque où les risques de crue sont faibles.

- inversément, à partir du mois d'août, rien ne s'oppose à ce que les électriciens turbinent et pompent plus de 6,5 hm3 en un jour, le maximum étant 17,15 (max. EdH) - 7,6 (min. EdH) = 9,55 hm3.

- cependant, compte tenu qu'au mois d'août et en début septembre, le risque de crue est faible mais que la saison touristique bat son plein, le Chef de Service des Barrages peut décider que, par dérogation au graphique des capacités optimales, le niveau de remplissage demeurera le plus élevé possible, sans dépasser toutefois 46,35 (EdH max. normal) + 29,7 = 76,05 hm3 pour le total des deux lacs principaux.

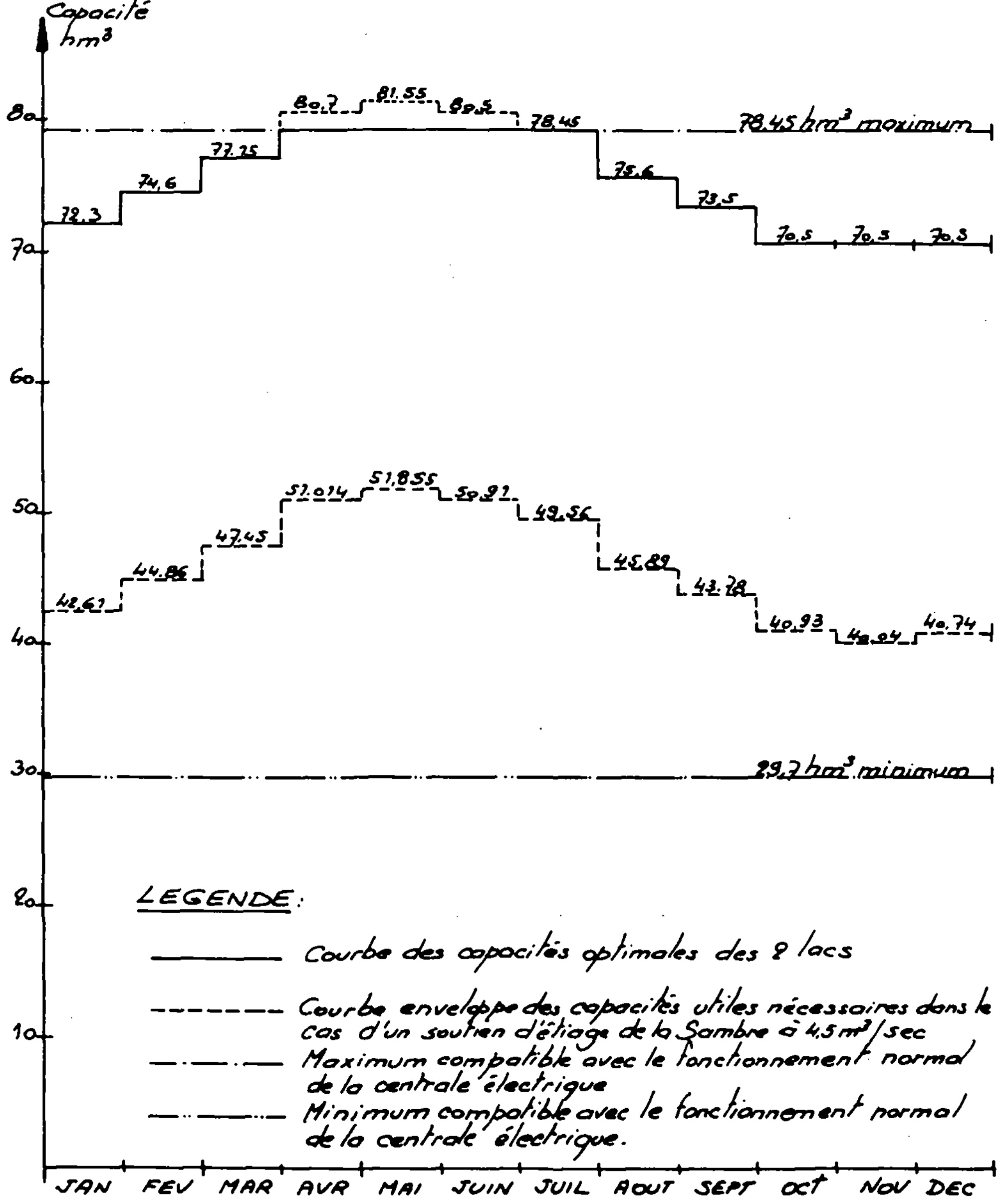
*

* *

Annexe 1

GESTION INTEGREE DU COMPLEXE DES BARRAGES DE L'EAU D'HEURE.

ETABLISSEMENT DU GRAPHIQUE DES CAPACITES OPTIMALES SUR BASE DE LA PERIODE 1969-1980.



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CHOOSING THE APPROPRIATE FORECASTING TECHNIQUE

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ABSTRACT

The potential savings from precision to estimating future urban water use are obvious. And, because of the need to predict the effectiveness of potential water conservation measures, new and more responsive approaches of disaggregated demand forecasts are mandatory.

The purpose of this study is to assess current water use forecasting practice in the U.S. Army Corps of Engineers and to recommend those additional approaches which best satisfy current requirements. To accomplish these objectives, this report presents the findings of a three-prong investigation: (1) identification of current needs for improved forecasting approaches in light of the current requirements; (2) review and assessment of current forecasting approaches; and (3) recommendation of the most appropriate forecasting approaches which meet the identified needs and satisfy current requirements. Data were obtained from personal interviews with field planners in 6 districts and 3 divisions, from a questionnaire to 35 districts and 11 divisions, and from the analysis of 27 Corps studies that had forecasted demand.

CHOOSING THE APPROPRIATE FORECASTING METHOD

Long range forecasts of future water use are indispensable to the efficient management of municipal water supply systems. Forecasts, and the planning activities which use them, determine the investments which water utilities make, or forego. Miscalculations in preparing or applying forecasts are always serious. They may result in the overcommitment of large amounts of money, diverting it from other critical needs, or, on the other hand, the failure of a water supply system to meet its demand. Although forecasting is admittedly a risky business, it is more important today that forecasts be made with greater accuracy, and that they be made with the best information and techniques available to reduce the cost of error.

This report examines current needs for improved forecasting approaches in light of current planning requirements; reviews and assesses current forecasting approaches; and recommends the most appropriate forecasting approaches to meet identified needs. It also includes a discussion of a highly disaggregated computer forecasting model known as the IWR-MAIN System.

Data for this study were obtained from personal interviews, from questionnaires to federal water planners, and from the analysis of 57 water supply studies that had forecasted water demand.

ASSESSMENT OF EXISTING APPROACHES

Many different approaches have been used or proposed for water use forecasting. Furthermore, specific forecasts may incorporate the use of several distinct approaches. In order to provide an evaluation of forecasting approaches, therefore, a limited number of prototypical methods were chosen for discussion and evaluation. Each prototype was evaluated according to the following criteria: (1) SCOPE - any limitations imposed by the approach: (2) DISAGGREGATION - suitability of the approach for use in preparing sectorally and geographically disaggregated forecasts; (3) MULTI-VARIATE MODELS - choice of explanatory variables used; (4) ALTERNATIVE FUTURES - ease of incorporating alternative assumptions regarding future conditions; (5) CONTINUITY ASSUMPTIONS - nature of the assumed underlying process as to its stability; and (6) COMPATIBILITY - approaches must be compatible with the planning process, field conditions, skill requirements, etc.

The prototypical approaches fall into six broad categories, with each category containing a range of specific techniques having similar characteristics. The categories are: (1) per capita methods; (2) per connection methods; (3) unit use coefficients methods; (4) multivariate requirements models; (5) multivariate demand models; and (6) probabilistic methods. The first three categories are all single coefficient methods: each employs a single explanatory variable. The fourth, and fifth categories are multiple coefficient methods, using more than one explanatory variable. The sixth category use probabilities in conjunction with one of the above methods.

<u>Forecasting Methods</u>

<u>Per capita coefficient</u> - is the most widely used water used water use forecasting method: future water use is estimated by multiplying expected future population by an extrapolated water use coefficient. The method is simple, requires little data, and the data are easily obtained. Yet, in spite of its popularity, this approach has serious shortcomings in the fact

that results are insensitive to most trends and changes known to affect urban water use, and provides minimal information to those wishing to plan future facilities or management strategies.

Per connection coefficient - expresses future water use as a product of the expected future number of customers connections and a per connection water use coefficient. The value of the coefficient is usually extrapolated from past experience. The major advantage of this method over the per capita approach is that historical data on number of connections to a water system is likely to be more readily available and more accurate than data on past population served. Also, number of connections is well correlated with aggregate water use.

Unit use coefficient - uses a single explanatory variable other than population or number of customer connections. Unit use coefficients differ from the per capita and per connection techniques, however, in that they are more often used in the context of sectorally disaggregated forecasts.

Multivariate requirements models - incorporate more than one explanatory variable and fall into one of two categories: those employing requirements models and those using econometric demand models. Requirements models include variables observed to be significantly correlated with water use, not necessarily those suggested by a priori economic reasoning. The number of explanatory variables may range from two to several dozen, and the dependent variable may be aggregate (total) or sectoral water use. Data requirements can be considerable, depending upon the number of explanatory variable used, and data are sometimes comparatively difficult to collect. Data collection efforts must be balanced against potential improvements in the accuracy and usefulness of the forecast.

Multivariate demand models - are based on economic reasoning, and include only variables which are expected to be causally related to and found to be significantly correlated with water use. Demand models consider price and income, as well as other variables. The number and nature of the explanatory variables actually used may vary greatly from one application to another, according to data availability, required accuracy, local conditions, etc.

Probabilistic methods - provide a means for considering uncertainty in a water use forecast. Ordinarily, this approach requires that a base forecast be prepared, using one of the methods mentioned above. Possible sources of uncertainty regarding future water use levels are identified, and subjective probabilities are estimated for each possible outcome. The base forecast is modified to reflect the effects of all possible combinations of the un- certain factors, one combination at a time, and the joint probability of each of the combinations is associated with the forecast water use expected to result from the combination.

All of these approaches are used to varying degrees. Some approaches enjoyed wide use and acceptance, others are applied in isolated cases, or have been adopted in the recent past.

The techniques were tabulated against the earlier described needs to determine which approaches best satisfied specific needs. Particular attention was given to approaches that have been applied under actual planning conditions. These evaluations applied to the capabilities of the forecasting approaches, and not the characteristics of the forecast which they might produce. The evaluations were necessarily of a summary nature, touching on major issues and considerations.

Table 1 presents a comparison of the needs and capabilities of the various approaches. Although actual forecasting techniques may differ from those shown in one or more details, the prototypical approaches represent realistic possibilities along the continuum of all possible forecasting approaches.

Table 2 summaries some general observations about the application characteristics of the seven forecasting approaches studied. It can be seen that consistency with the requirements of the U.S. Water Resources Council's Principles and Guidelines for Planning Water and Related Land Resources, and with the need to evaluate possible water conservation measures, is best obtained by appropriate application of unit use coefficient methods, multiple coefficient methods, and the probabilistic method. On the other hand, the three simplest forecasting approaches, plus the unit use coefficient method are likely to remain a part of various preliminary planning efforts. The small quantities of data required, and the ease of obtaining such data, argue for their continued use in appropriate applications.

NEED FOR IMPROVED APPROACHES

Water use forecasts are employed in a wide variety of planning studies conducted. Forecasts are frequently used by the Federal Government in planning the water supply purpose of a multiple purpose water resource project. Federal engineers also use water use forecasts to reallocate a major impoundment. Most forecasts are used by water utilities as the basis of design of water supply facilities and are of the longer-range type of projections.

The most commonly employed forecasting technique today relies on an aggregate description of water use, which is a forecast based on a single use coefficient (usually water use per capita) whose value may or may not be permitted to change during the forecast period.

Aggregate forecasts, however, are insensitive to changing sectoral patterns in developing communities, including differential growth rates for multi-unit and single-unit housing. Specific water conservation measures, which often selectively alter water use by user sector, is difficult to consider with the absence of sectoral disaggregation.

Since most variables known to affect water use are omitted (such as price, income, family size, irrigable area, weather, levels of commercial and institutional activity, etc.), aggregate forecast are insensitive to any changes from past relationships existing among these variables. In particular, the sensitivity of future water use to alternate planning assumptions cannot be determined.

All of these things necessitate the need to change the way water use forecasts are prepared.

It should be noted, however, that attempts to develop disaggregated forecast have been hampered by the general inability of water utilities to produce the analyses of billing records needed to support the development of the necessary forecasting models. Further, the inclusion of additional explanatory variables creates the requirement to forecast future values for those variables, multiplying data requirements in areas where data may not be readily available.

CONCLUSIONS

This survey disclosed a wide range of forecasting methods actually in use, including all approaches defined above except the probabilistic method. Some approaches now in use are not widespread, but they have been applied.

Generally forecasting approaches now in use do not facilitate disaggregation by user sector, or by season and are not usually multi-variate, as they depend primarily on population to explain future water use. Alternative futures is not assisted by current methods and most underlying assumptions are implicit and may not be known to the planner or the intended audience. Current methods rely heavily on the assumption of continuity of past trends, and provide relatively little information to the planning process.

In addition, the two areas were identified where additional experience would be helpful. The first is the forecasting approach which introduces a probabilistic dimension to water use forecasts and greatly increases the amount of information which the forecast can convey to the planning process. The second area for further development is the use of flexible, computerized forecasting systems which allow an array of forecasting methods, combined with necessary data management procedures.

Computerized Forecasting System

An existing computerized forecasting system developed by Hittman Associates, Inc. (1969), known as the MAIN II System, has been modified by the U.S. Army Corps of Engineers Institute for Water Resources for use by its field planners and anyone else who wishes to use the model.

The IWR-MAIN System considers four major sectors of water use: (1) residential; (2) commercial/institutional; (3) industrial; and (4) public/unaccounted users. Each sector is further disaggregated as needed for forecasting purposes. As many as 284 individual water use categories can be used. Water use models included in the IWR-MAIN System include econometric demand models (residential categories), unit use coefficient models (commercial/institutional and industrial categories) and per capita models (some public categories). The structure of the IWR-MAIN System is shown in Table 3.

For each future time period, water use is calculated as a function of a set of projected parameters or explanatory variables. Four alternative methods are available for predicting future levels of these parameters: (1) projection by internal growth models; (2) projection by extrapolation of local historical data; (3) use of projections made external to the model and provided as input by the user; and (4) any combination of the above.

The IWR-MAIN System is capable of preparing highly disaggregated forecasts of future water use, based on a moderate quantity of good quality data. This is especially true of base year data, where the greatest detail is required. Inaccuracies in base year data may alter subsequent forecasts in complex and subtle ways, not readily apparent to the analyst.
This forecasting program has received wide application in utility contexts and was tested on independent sets of data. Retrodictions or "backcasts" of water demand by customer class were generated for communities based on known historical values of explanatory variables. The results are

presented in Table 4.

More recently, the model was tested against two cities with the results generally the same as before even though water use characteristics and trends have changed over the last twenty years. These results are presented in Table 5.

The IWR-MAIN System is currently undergoing some dramatic changes and no longer resembles its earlier brother. All of the current water use models have been revised to more accurately reflect today's water use patterns. Next, the growth (or projection) models are revised. Plans are also underway to incorporate water conservation evaluations into the model.

The program is available for mainframes as well as IBM PC's. The microcomputer version is a greatly enhanced version and easier to use than the mainframe version.

SUMMARY

Traditional forecasting procedures, for the most part, were developed during a period when water supply planner's responsibility did not extent beyond the provision of water supply storage. However, changes in the world economic environmental and social policy, as well as needs to make utility systems more reliable, stimulated increased concerns to improve water supply planning techniques. Planners are required now to apply new forecasting techniques, and to collect types of data that have not been required in the past in order to reduce the cost of forecast error. Forecasting methods are destined to become more complex and sophisticated at the same time accuracy becomes more critical. Tools, such as the IWR-MAIN System, are now becoming available to aid the analyst in providing a more accurate and reliable forecast.

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TABLE 1
Comparison of Existing Approaches to Planning Needs

Planning Need	Simple Time Extrapolation	Single Coefficient Methods: Per-Capita	Single Coefficient Methods: Per-Customer	Single Coefficient Methods: Unit Use Coefficient	Multiple Coefficient Methods: Requirements	Multiple Coefficient Methods: Demand	Multiple Coefficient Methods: Contingency
Permits prediction of various measures of water use	+	+	+	+	+	+	+
Suitable for medium range forecasts	+	+	+	+	+	+	+
Suitable for long range forecasts	-	+	+	+	+	+	+
Facilitates sectoral disaggregation	-	-	+	+	+	+	n.a.
Facilitates geographic disaggregation	-	+	+	+	+	+	n.a.
Includes adequate explanatory variables	-	-	-	-	0	+	+
Requires reasonably available data	+	+	+	+	0	0	0
Provides detailed planing information	-	-	-	-	0	+	+
Demonstrated under existing conditions	+	+	+	+	+	+	n.f.

Legend: + - Yes; 0 - Unknown; - - No; n.a. - not applicable; n.f. - not found in survey

TABLE 2
Application of Forecasting Approaches

		Single Coefficient Methods			Multiple Coefficient Methods		
Application Characteristic	Simple Time Extrapolation	Per Capita	Per Customer	Unit Use Coefficient	Requirements	Demand	Contingency
Facilitates forecasts consistent with Principles and Guidelines	no	no	no	when used in disaggregate forecasts	when used in disaggregate forecasts	when used in disaggregate forecasts	yes
Facilitates evaluation of water conservation measures	no	no	no	"	"	"	yes
Suitable for reconnaisances type studies	yes	yes	yes	yes	no, too complex	no, too complex	no, too complex
Suitable for detailed planning	no	no	no	when used in disaggregate forecasts	when used in disaggregate forecasts	when used in disaggregate forecasts	yes
Data Requirements:							
quantity of data needed	very little	very little	very little	moderate	moderate to large	moderate to large	depends on application
difficulty of obtaining needed data	low	low	low	low to moderate	moderate to high	moderate to high	depends on application
Adequacy of training and experience of field planners	adequate	adequate	adequate	adequate	further training required	further training required	further training required

TABLE 3
Internal Structure of the IWR-MAIN Model

Sector	Category
Residential	Metered and Sewered Domestic Use (winter and summer) Mastered Meter Apartments Domestic Use Flat Rate and Sewered Domestic Use Sprinkling Use[a] Flat Rate with Septic Tanks Domestic Use Sprinkling Use[a]
Commercial/ Institutional	Subdivided by type of establishment: up to 50 categories available -28 categories provided
Industrial	Subdivided by 3- and 4-digit SIU Code: -200 categories provided
Public/ Unaccounted	Subdivided by type of water use: up to 30 categories available -2 categories provided

[a]Separate sprinkling water use models provided for east and west U.S.

TABLE 5
Recent Application Experience with the IWR-MAIN System

	Actual Demand (MGD)	IWR-MAIN Estimate (MGD)
Chester, Pennsylvania (1980)		
Residential	5.0	7.52
Commercial/Institutional	1.45	3.20
Industry	12.02	11.13
Public/Unaccounted	5.24	2.05
Total	23.71	23.85
Springfield, Illinois (1980)		
Total	19.7	19.6

TABLE 4
Application Experience with the IWR-MAIN System

	Actual Demand (MGD)	IWR-MAIN Estimate (MGD)
Baltimore, Maryland (1963 data)		
Residential	97.3	95.2
Public-Commercial	19.6	19.2
Industrial	42.0	45.1
Total	158.9	159.5
Park Forest, Illinois (1959 data)		
Residential	1.68	1.49
Commercial	0.15	0.15
Park Forest, Illinois (1961 data)		
Residential	1.70	1.58
Par, Forest, Illinois (1962 data)		
Commercial	0.18	0.15
Park Forest, Illinois (1963 data)		
Residential	1.72	1.58
Commercial	0.19	0.17
Park Forest, Illinois (1965 data)		
Residential	1.91	1.75
Commercial	0.21	0.19
Park Forest, Illinois (1967 data)		
Residential	1.91	1.84
Commercial	0.21	0.20
Baton Rouge, Louisiana (1965 data)		
Residential	n/a	14.0
Commercial	n/a	5.20
Public & Unaccounted	n/a	4.36
Total	23.8	23.6
Kings Heights District Anne Arundel County, Maryland (1968 data)		
Residential & Commercial	0.31	0.32

n/a - not available

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THE COLLECTIVE PROVISION OF POTABLE WATER TO RURAL AREAS: ORGANIZATION, OPERATING COST AND DEMAND EVIDENCE FROM A MAJOR U.S. CORNBELT STATE, U.S.A.

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ABSTRACT

Substantial capital has been provided by the U.S. federal government to constuct water supply systems in the open countryside and small communities of rural America. Authorized in 1937, the capital subsidy programs initially focused on the western arid regions of the nation. Subsequent changes expanded the program to all rural regions of the U.S. This paper reports a study of the U.S. government financed rural water systems meeting the domestic water demands of farm and nonfarm residents in Illinois, a major cornbelt state in the U.S. midwest, a region not traditionally viewed as having major water problems. The study focuses on the 59 systems that do not service any towns or villages and whose customers are farmers and other residents of the open farm country. Reported are analyses of the system's operating status, and costs. Rural water service demand is analyzed using rural water system customer data and a pooled time series, cross-sectional econometric model. Theoretical consideration is given to the implications of demand analysis of goods, like water, priced using declining block rate schedules. Own price and income elasticities for rural water services are reported. These elasticities are useful in managing water supplies, the quantity of water demanded and water system revenues. Management policies and fiscal soundness have become more important in the 1980's as U.S. federal government policies under the Reagan Administration have reduced the availability of subsidized capital.

INTRODUCTION

Rural water systems provide potable water to residents in many rural areas in the United States. Encouraged by financial construction subsidies from the U.S. federal government, water systems serving the domestic water demands of farmers and other residents of the open countryside can be found in every region of the country. Rural water systems generally include a water source (ground or surface), pumps, pipelines, storage facilities, treatment plants and ancillary facilities and equipment required for the procurement of water supplies and delivery of potable water to users.

This paper presents a study of the 59 rural water systems in Illinois, a major agricultural and industrial state in the corn belt of the United States. The organization and instituitional characteristics of rural water systems are presented first followed by a description of the systems and an analysis of costs. An overview of rural water system customers is next including analysis of rural water service demand.

LEGISLATIVE HISTORY

U.S. government policy on rural water programs was initially codified in 1937 when the U.S. Secretary of Agriculture was authorized to make direct long term, low interest loans to assist in the construction of facilities for water utilization and storage and to develop water facilities for household and farm use. Assistance was confined to the seventeen arid and semi-arid western U.S. states. In 1954 Congressional action extended the authorization to the entire U.S. The passage of the Consolidated Farmers Home Administration Act of 1961 initiated an era of rapid expansion of U.S. government credit activities for water service and other rural development programs. Construction grants of up to 50 percent of the development cost of projects were authorized in 1965.

As a national policy, the 1965 amendments recognized the special need of rural areas for water supplies. The objective was to provide rural areas with water service, comparable to that available in cities without regard to whether the rural areas were primarily associated with agriculture (Chicoine, Grossman and Quinn 1984).

The most recent major statutory change to the rural water system programs was in 1972. Grants may now cover up to 75 percent of project development costs and the statutory definition of rural was extended to include towns and villages of not more than 10,000 population. Grant eligibility and the interest charge on loans are now need tested. Grants are used for water systems serving the most financially needy areas to reduce the debt service portion of annual average user costs.

The interest rate for each loan is established on the date of loan approval and set at least each quarter of the fiscal year based on the U.S. governments marginal cost of money in the bond market and the median family income in the service area. A five percent fixed rate is available if the median income is below the U.S. government poverty line and the financing is needed to meet health standards. The market rate of interest is charged if the median family income of the service area is more than 85 percent of the rural median income level. An intermediate rate is used if service area family incomes are not more than 85 percent of the rural median income.

U.S. government domestic policy redirection, beginning in 1980, has reduced substantially available grants. The historical five percent capital charge over a 40 year loan repayment period has been replaced by the market based need tested sliding interest rates. These policy changes increase the cost of rural water services to users.

ILLINOIS RURAL WATER SYSTEMS

U.S. government financial assistance exceeding $109 million has been obtained by rural water districts and small towns for water services in Illinois. Of the over 250 local agents receiving federal government financial assistance, 59 are rural water districts providing water only to farmers and rural residents outside towns and villages. The 59 rural systems serve approximately 24,000 customers and maintain about 4,200 miles of water line. Data on the rural systems were collected from a mail survey distributed to the systems and from expenditure information available in the files of the federal rural credit agency.

Rural Water District Costs

Water services have traditionally been treated as largely self supporting enterprises not subsidized with local taxes. These services, for the most part been paid for by users. To be generally self supporting in the long run, enough revenue must be collected from water sales to cover costs. All costs must be considered, whether variable or fixed. For water services, three categories of costs can be delineated: 1) variable operations costs, 2) semi-fixed or general and 3) fixed capacity costs (Hirshleifer, DeHaven and Milliman 1969).

As capacity costs and semi-fixed costs are spread over more units of water, the average total costs of water service are expected to decline steadily over the entire range of feasible output. This is the classic case of a decreasing cost industry with marginal cost lying below average cost within the feasible range of output (Afifi and Bassie 1969). Marginal cost pricing, as an efficient pricing rule, results in operating losses since total revenues do not cover total costs. These conditions lead to a two-part pricing system applied through block water rate schedules.

The requirement that all rural water system costs, including capital and operating and maintenance costs, must be considered can be conveniently expressed as:

$$C = OE + ER + (DS - S) \tag{1}$$

where: C is the total annual cost, OE is the variable operating and semi-fixed expense and ER + (DS - S) is the annual capacity costs comprised of equipment depreciation (ER) and net annualized capital costs (DS - S) or debt service. DS is the gross annualized debt service and S is the annualized adjustment accounting for front-end capital contributions. These front-end contributions, which reduce initial borrowing (increase S) include: 1) "user benefits" which are front-end monetary or in kind customer contributions (e.g. tap-in or membership fees) and/or 2) subsidies such as U.S. government construction grants. The higher the interest rate on borrowed capital, the greater is DS (Ramamurthy and Chicoine 1984).

Operating Cost Analysis

Descriptive evidence suggests Illinois rural water systems are subject to economies of size. However, other factors, such as customer density may inhibit the realization of these economies. To isolate the factors contributing to the variation in expenditures per unit of water sold, a statistical cost analysis of the average operating cost of rural water services was conducted (Chicoine, Ramamurthy and Grossman 1984).

The investigation of the operating cost of water services involves either 1) the examination of the economic behavior of one production unit or system over a number of years or 2) the analysis of a number of systems in a specific time period. The later method was used here since available data are cross- sectional. The following generalized model was estimated:

AUC = f(QNTY, QNTY , USRS, TRTCST, PRCH, TOPO, CLMT, EFF, WAGE)

where:

AUC = average operating cost per million gallons (PMG) of water per year,
QNTY = the quantity of water supplied in million gallons per year,
USRS = number of users or number of users per mile of pipe,
TRTCST = treatment costs PMG of water supplied in standardized form,
PRCH = water purchased measured as a percent of water supplied,
TOPO = 1 for rough topography, 0 otherwise,
CLMT = county average annual percentage of frost free days,
EFF = system efficiency measured as a ratio of water supplied to water produced, and
WAGE = salary outlays PMG of water supplied in standardized form.

The general cost function was estimated in linear form using OLS regression analysis. The results are reported in Table 1. The model performed reasonably well, accounting for about 70 percent of the variation in average operating cost. Equations 1 and 2 include quadratic variables for QNTY and number of users (USRS). The significant and oppositely signed coefficients for QNTY and QNTY suggests a U-shaped relationship between cost and output.

The positive significant coefficient on USRS in equations 1 and 2 indicates that an additional user adds about $1.40 to the operating costs of providing one million gallons of water. While this finding is consistent with Daugherty and Jansma (1973), no adjustment is made in this specification for the spatial dimension of water services. To investigate the relationship between user density and average operating cost, the cost model was estimated with USRS scaled to miles of pipeline. These results are reported in equations 3 and 4.

The coefficient on USRS/PIPE MILES is negative and significant at the five percent level. In general, for every additional user per mile of pipe, average operating costs declined by about $35 PMG of water. As evident in equation 3, the scaling of USRS by the miles of line reduced the size and significance of the coefficients on QNTY and QNTY . Also, the TOPO and PRCH's coefficient became insignificant (equation 3).

Table 1. Statistical Operating Cost Analysis of Illinois Rural Water Systems

Variable	Equation 1	Equation 2	Equation 3	Equation 4
QNTY	-36.53 (2.45)*	-37.49 (-2.54)*	-17.16 (1.21)	-8.94 (2.03)*
$QNTY^2$	0.17 (1.99)*	0.17 (2.02)*	0.08 (0.55)	---
USRS	1.41 (2.29)*	1.45 (2.37)*	---	---
USRS/PIPE MILES	---	---	-33.88 (2.51)*	-34.55 (2.99)*
TRTCST	436.81 (5.26)*	438.39 (5.30)*	459.28 (4.64)*	462.90 (4.90)*
WAGE	559.37 (6.25)*	560.19 (6.29)*	324.75 (1.84)**	316.09 (1.95)**
EFF	-21.65 (3.17)*	-20.68 (3.10)*	-17.77 (2.02)**	-15.46 (1.84)**
TOPO	-238.94 (1.36)	-293.41 (1.86)**	-231.25 (1.08)	---
CLMT	-25.21 (0.72)	---	---	---
PRCH	9.10 (2.50)*	9.19 (2.53)*	2.57 (0.46)	---
Constant	5,233.17 (2.64)*	3,871.71 (6.23)*	4,381.18 (5.80)*	4,024.93 (5.94)*
N	62	62	45	45
Adj R^2	0.737	0.740	0.668	0.669
F	20.03*	22.67*	13.64*	18.81*

a. Numbers in parentheses are the absolute values of the t-ratios.

* Significant at the 5% level.

** Significant at the 10% level.

The inclusion of the spatial aspects of rural water services raises questions about the U-shaped relationship between cost and output reported in equations 1 and 2. When the quadratic variable for QNTY was excluded from the model along with other variables shown to be not significant (equation 4), no diseconomies of scale are evident over the size range of Illinois systems studied. This suggests that per unit operating costs may be reduced with larger scale systems, perhaps through regional service provision.

RURAL WATER SYSTEM CUSTOMERS AND DEMAND

Water has become an economic good. The technology is now available to collectively provide potable water on demand to scattered farms and rural residences outside towns and villages. It is a matter of economics and how much customers are willing to pay for water at their location. To present a profile of the customers of Illinois' rural water districts and an analysis of the demand for rural water services, a survey of a stratified random sample of customers households was conducted. Through a telephone interview, surveys were completed on 100 randomly selected customers. The interview data was matched with monthly household water consumption and expenditures for 1982 collected from rural water district records. In general, the average Illinois rural water district customer would likely be a nonfarm rural resident, report a 1982 income of about $17,000, be 49 years of age, be a household containing 3 or 4 persons, consume 4,640 gallons of water in an average month at an average price of $5.77 per 1,000 gallons and pay a water bill of $20.47 per month, on average. Only about 22 percent of the customers reported their occupation as farmers.

Water Demand

Determination of the demand for rural water is needed to assist the boards of rural water districts 1) manage water supplies--water use forecasts are important in evaluating new supply facility investments, 2) manage water demand--estimate the impact of policies such as price changes and non-price rationing regulations of water use, and 3) manage revenues--evaluate the impact of rate changes on receipts. Little research has been undertaken that focuses on rural water demand and several previous studies fail to account for the block rate system of pricing water services (e.g., Doeksen, Goodwin and Oehrtman 1984); Hanke and de Mare 1982; and Foster and Beattie 1979).

More recent studies of the demand for residential water have based their analysis on a model of consumer behavior suggested by Taylor (1975) and modified by Nordin (1976) (Billings and Agthe 1980, and Hanke 1982). Here the quantity purchased by a consumer through a block rate pricing schedule is expressed as a function of the marginal price faced by the consumer and a second price related factor defined as the difference between the consumer's actual bill and the product of the marginal price and the amount of water purchased (Nordin 1976). The second factor reflects the income effects associated with the discreet changes in the water rate schedule. Nordin's analyses, which was a theoretical treatment and considered only decreasing block rate schedules was extended by Billings and Agathe (1980) to include increasing block rate schemes. There is also a suggestion that the common practice of using aggregate data, where the quantity demanded and income variables are averages over some unit of observation, have resulted in theoretically inconsistent estimates of demand (Schefter and David 1983).

Using the consumption and expenditure data from the sample of Illinois rural water district customers, a multiple tariff demand for rural water services was estimated. Demand elasticities for water under the system of declining block rates in these districts were derived.

Empirical Demand Estimates

The survey data from Illinois rural water district customers were matched with monthly consumption and expenditure data from the records of the rural water districts. The water rate schedules from the districts were used to determine marginal prices (P) and to calculate difference variables (D). Missing observations caused the sample size to fall to 77. The consumption and expenditure data are monthly for 1982 so the data consists of a time series over 12 periods across the 77 rural water district customers for a total 924 observations. The demand for rural water was estimated using the following pooled time series and cross-sectional model:

$$Q = \sum_{t=1}^{11} B_{1t}M_t + B_2RES_i + B_3ALTSRC_i + B_4DISH_i + B_5NUMBRES_i + B_6BATH_i +$$

$$B_7P_{it} + B_8D_{it} + B_9INC_i + e_{it}$$

where:

Q_{it} = water consumption by houseshold i in month t,
M_t = monthly binary variable where t = 1 for Jan., 2 for Feb.,.., 11 Nov.,
RES_i = type of residence binary variable with farm = 1, 0 otherwise,
$ALTSRC_i$ = existence of an alternative onsite source of water, 1 if yes; 0 otherwise,
$DISH_i$ = 1 if household i has dishwasher, 0 otherwise,
$NUMBRES_i$ = household size measured by number of persons,
$BATH_i$ = number of bathrooms in household i,
P_{it} = marginal price paid by i th household in month t,
D_{it} = difference between household i's water bill in month t and Q_{it},
INC_i = monthly income of household i, and
e_{it} = random vector distributed as N(0, I).

Using ordinary least squares, the demand for rural water services was estimated, employing only those observations where consumption was beyond the first block in the rate schedule. In the first block P_{it} = 0 and the customer pays the minimum charge with the right to consume a given quantity of water. This reduced the number of observations to 797.

Because of the pricing policies followed by rural water districts and other types of utility enterprises, marginal price may not be exogenous in the long run. The rate schedule, over time, is dependent on the volume of water sold. In establishing rates, water sales will be considered. However, to individual customers in the short run, price will be exogenous. Consumptive behavior over short period will not cause policies to be altered and rates increased. During the 12 months over which expenditure and consumption data were collected, no rural water district altered its rates. Thus, for the purposes of demand estimation, the price of water can be assumed exogenous.

The empirical estimates of the demand for rural water are presented in Table 2. The signs on the coefficients of household income, price, household size, and number of bathrooms are as expected and significantly different from zero. The coefficients on the dishwasher, farm/nonfarm and other domestic water source binary variables are not statistically different from zero. These three variables were excluded from the estimates reported in Model 3 and 4. As suggested by Judge et al., an F-test is used to test for time series effects or if a seasonal pattern exists in water consumption. The calculated F-ratios for Model 1,2,3 and 4 are 1.34 and 1.35, respectively. The critical value at the 95 percent level for the respective calculated values are 1.83 and 1.75. The critical value of the F-ratio did not allow the rejection of the null hypothesis that there are no time series effects. Thus, for estimation purposes the observations can be treated as one sample.

Table 2. Estimated Rural Water Service Demand[a]

Dependent Variable	Model 1	Model 2	Model 3	Model 4
Household Income (av./mo.)	.0006 (5.39)*	.0006 (5.34)*	.0005 (5.43)*	.0005 (5.35)*
Marginal Price	-.5853 (12.53)*	-.5927 (12.67)*	-.5757 (13.17)*	-.5822 (13.33)*
Difference	.0126 (0.65)	.0153 (0.79)	.0166 (0.89)	.0192 (1.05)
Household Size	.5995 (9.13)*	.5938 (9.03)*	.6114 (9.50)*	.6056 (9.39)*
Number of Bathrooms	1.1927 (8.51)*	1.1930 (8.67)*	1.1242 (8.75)*	1.1193 (8.68)*
Dishwasher 1=yes, 0 otherwise	-.3057 (1.33)	-.3141 (1.40)	—	—
Farm/Nonfarm 1=farm, 0 otherwise	-.1188 (0.42)	-.1129 (0.40)	—	—
Other Water Source 1=yes, 0 otherwise	.0510 (0.24)	.0471 (0.22)	—	—
M1, 1=January 0 otherwise	.0809 (0.18)	—	.0806 (0.18)	—
M2, 1=February 0 otherwise	-.0771 (0.17)	—	-.0801 (0.18)	—
M3, 1=March 0 otherwise	.5067 (1.17)	—	.5101 (1.17)	—
M4, 1=April 0 otherwise	-.2012 (0.46)	—	-.2002 (0.46)	—
M5, 1=May 0 otherwise	.3825 (0.89)	—	.3845 (0.90)	—
M6, 1=June 0 otherwise	.6286 (1.43)	—	.6163 (1.65)	—
M7, 1=July 0 otherwise	.7894 (1.84)**	—	.7871 (1.83)**	—
M8, 1=August 0 otherwise	.5730 (1.34)	—	.5793 (1.34)	—
M9, 1=September 0 otherwise	.6332 (1.48)	—	.6345 (1.48)	—
M10, 1=October 0 otherwise	-.1761 (0.41)	—	-.1773 (0.41)	—
M11, 1=November 0 otherwise	.0633 (0.15)	—	.0636 (0.15)	—
Constant	2.1482 (4.18)*	2.4609 (5.87)*	2.0858 (4.17)*	2.4018 (5.98)*
Adjusted R^2	.41	.41	.41	.41
F	30.18	69.49	35.76	110.90
RSS	4761	4851	4773	4864
N	797	797	797	797

a. Dependent variable is the quantity of water purchased per month in thousands of gallons. The absolute value of t statistics are in parenthesis. RSS=residual sum of squares.

* Significant at the .05 level.

** Significant at the .10 level.

Theory suggests the coefficients on Difference and Household Income should be equal in magnitude and opposite in sign in a linear demand model since each measures a pure income effect. The coefficient on the difference variable reported in Table 2 is positive, which is opposite that expected, but it is not statistically different from zero. An insignificant coefficient on the difference variable has been reported in other studies (Howe 1982). One explanation for the insignificant coefficient on Difference is the low proportion of income it absorbs. On average, the difference variable absorbs .34 per income which may be too small to have any significant impact on consumer's perception of income.

In addition, Foster and Beattie (1979) present arguments and some evidence that challenge the appropriateness of the marginal price difference demand model and its perfect knowledge postulate in analyzing consumer price response behavior for potable water.

The adjusted R for the estimated demand in Model 4 is .41. The estimated elasticities of rural water demand for own price, income and household size are -.39, .18 and .37 respectively. The price elasticity is similar to elasticities reported in studies of rural and urban water demand (e.g., Doeksen, Goodwin and Oehrtman 1984; Billings and Agathe 1980). The insignificant coefficient on Difference in Table 4 indicates the impact of price changes over the range of the rate schedules studied are described entirely by the marginal price elasticity. The elasticity of -.39 indicates an inelastic demand for water at the mean of Q_{it} and P_{it}.

Demand Analysis Implications

Using Illinois rural water district customer data, a demand for rural water service was established. While the marginal price had the expected negative significant sign on its coefficient, the coefficient on the second price variable was not significant and had an opposite sign than expected. These results may be associated with the small proportion of household income accounted for by the difference price variable. Rural water demand was shown to not exhibit any seasonal variation. The estimated price elasticity of demand for rural water was -.39 while the income elasticity was estimated to be .18. The price elasticity suggests that by increasing the price of water, a rural water system will increase its total revenues. Also, for the range of water prices studied ($5.00 to $1.25 per 1,000 gallons), a price change will not result in a dramatic change in quantity of water demanded. Thus, small price changes will not be effective policies in allocating a short water supply.

SUMMARY

U.S. federal government policies have stimulated the growth of local agents that provide potable water to farmers and other residents of rural areas and small towns and villages. The stimuli come from capital assistance, through loans or grants, for project construction. Initially authorized in 1937 for arid regions of the country, policies were made geographically inclusive in the 1960's. The financial assistance, which is need tested, is used by eligible local agents to construct and operate waterworks systems. In Illinois the local agents are commonly special district units of government formed through voter petitions and popular referenda.

In Illinois, public water supplies are provided to farmers and others outside cities by 59 rural water districts. Generally the districts are small, and have low customer density. Cost evidence suggests technical efficiencies exit in the provision of rural water services. However, low customer densities complicate the collective provision of potable water in rural areas. The demand for rural water was shown to be generally inelastic over the price range observsed in Illinois rural water districts districts. The elasticity coefficeint of -.39 is reasonably similar to the price elasticity of demand for urban water services.

The institutional and economic experience of rural water service systems in the U.S. provide some insight into how the water demand of those outside cities have been addressed in part. While the linkages of the U.S. federal system are not universally present in other political and social settings, the response of local agents to take advantage of national policy and successfully operate waterworks in response to local demand may be an applicable concept transferable to other appropriate areas.

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PUBLIC WATER MANAGEMENT INITIATIVES: THREE CASE STUDIES

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ABSTRACT

Water problems and the institutions that manage water are complex. Water issues must be understood within a complex framework of social, political, legal and economic constraints which affect all levels of government and the private sector. Yet it is increasingly clear that water problems must be solved through more effective management and use of the organizations that citizens, bureaucrats, politicians, and business people have designed and will continue to create.

Structural and technical approaches cannot solve these problems.

Change must occur in the way water resources are organized, governed and managed. If the structure exists from the "top down" necessary to effect that change, it is either ineffective or inoperative.

The growing public concern for water issues is supportive of grass-roots efforts to develop coalitions that cross bureaucratic, political and geographic boundaries.

Three such new coalitions have developed in the United States over the past two years.

If the world's water problems could be solved without regard for geographic or political boundaries, social customs, agencies, laws, economics and politics, then technical solutions might be the order of the day. But water problems are people problems, tied to the way we perceive and use our land and water resources. Technology, alone, will not solve them. Solutions must be socially acceptable, politically viable and economically feasible (Viessman, 1982).

Public perceptions of a resource help determine how a society uses that resource. Current water problems are, in many ways, the result of past misperceptions. For example, the standard practice of burying solid and hazardous wastes in landfills is based more on the public perception, "If you can't see it, it's not a problem," than on the geology of waste disposal and groundwater movement. A U.S. Environmental Protection Agency study conducted from 1972 to 1980 revealed that 30% of the nation's 181,000 waste disposal facilities were sited above usable aquifers on thin or permeable soil.

The idea that dilution is the solution to pollution was based on the belief that water is a limitless resource with infinite possibilities for neutralizing waste. The perception never took into consideration the fact that more pollution going into the same solution lessens the dilution, so that high levels of organic and inorganic compounds can now be detected in many American water supplies.

There is a pervasive public attitude in the United States that water exists in unlimited supplies and everyone can have as much as they want, even if they live in a desert environment. Three of the five fastest growing cities in the country, San Diego, Los Angeles and Phoenix (Holdrich, 1984), exist in an arid region with questionable future water supplies. Residents of Phoenix use an estimated 260 gallons of water per person per day, while residents of Tucson, 100 miles to the southeast, use 160 gallons of water. Tucson is purported to have a desert mentality while Phoenix has an oasis mentality (Alexander, 1984).

These attitudes and public perceptions have provided the basis for many of our strategies for managing water quality and water quantity, even though they do not reflect the reality of water supplies in the United States. They constitute social barriers to effective water management. But there are also political and economic barriers.

Political constraints to developing sound management practices include the bureaucratic maze of agencies with some form of water management authority. In the U.S. Congress alone, there are 12 committees and 18 sub-committees which address water issues, along with 20 federal agencies. Individual states have innumerable others, and often the lines of authority overlap or conflict, thereby impeding effective water management.

Frequent administrative turnover in water-related agencies further slows the management process. Since 1968, when the environmental movement began to formalize, there have been five U.S. presidents, eight Department of Interior secretaries, four EPA administrators and 183 different state governors, an average turnover of 3.7 per state. This kind of administrative turnover hinders long-range planning and program continuity essential to effective water management.

There are also economic constraints to sound management practices. Perhaps the foremost is the economic tenet that water is free. While every other resource has a clear market value (except air and soil), water is public domain. Municipal water bills reflect the cost of pipes and maintenance, but not a "per-gallon," in-source value for water. Water prices have been kept artificially low, thus providing no economic incentive to conserve or reuse.

Legally, the ultimate result of the doctrine of prior appropriation, the basis of water law in the West is the overuse of water in an arid region. One of the basic tenets of prior appropriation is that allocation is based on use. If the water is not used, rights to it are lost. This has often encouraged farmers to flood their fields in order to retain rights to the same allocation the following year.

The potential water crisis is an institutional crisis. Our social, political, economic and legal institutions do not reflect the reality of a limited water resource. The world is not running out of water. Neither did the world run out of energy with the oil embargo of 1973. It did, however, lose the ability to get as much oil as it wanted, when it wanted it, at the price it wanted to pay. Our social, political and economic institutions could not adjust to these new realities fast enough, and a "crisis" occurred.

The same is true with water. Declining water quality throughout the United States from groundwater contamination and the tremendous overdrafting of groundwater in the West is lessening the amount of usable, available supplies. Individuals, cities and regions may no longer be able to get as much water as they want, when they want it, at the price they want to pay. The energy crisis became manageable when people changed their perceptions and lifestyles to more adequately reflect the reality of a limited supply. The same will be true for water.

A NEW ERA

The current trend in national water management is to return increased responsibility to the states, but this responsibility has not been coupled with corresponding federal financial assistance. The current federal deficit will change the face of American water policy dramatically and irrevocably, requiring new financing initiatives between the public and private sectors and forging new political arrangements. These trends toward decentralized responsibility and local financing, together with increasing competition for usable supplies, will force changes in how water is used, allocated, priced and managed. These changes will test the resiliency of our social, economic, political and legal institutions.

Water management is the key to solving today's water problems and heading off those of tomorrow, and management is dependent upon the organizations that citizens, politicians, bureaucrats and business people have designed and will continue to create. Institutions are changing and must continue to change to meet emerging needs.

The growing public concern for water issues is supportive of grass-roots efforts to develop coalitions that cross bureaucratic, geographic and political boundaries. Three new management efforts have developed in the last two years in an effort to overcome institutional barriers to water management and to reflect decreasing federal authority and financing of water programs.

THE RED RIVER VALLEY PROJECT

During this transition from federal to state responsibility for water management, the question arose - "Are individual states prepared to fully manage water resources which often travel far beyond their political borders?" To test that question, a research program was initiated in the Red River Valley of the North, an area encompassing portions of North Dakota, South Dakota, Minnesota and the Canadian provinces of Manitoba and Saskatchewan. Funded by the Freshwater Foundation, the Ford Foundation, the University of Minnesota's Hubert Humphrey Institute of Public Affairs and the Minnesota Association of Counties Foundation, local organizational structures and governmental processes were analyzed to see if there was interest in regional water management and structures adequate for the task.

The Red River Valley was appropriate for such an effort because it is a microcosm of the complex water supply problems found nationally. In addition, the Red River is an interstate, intrastate and international resource, thus complicating jurisdiction over water management. It is one of the richest agricultural areas in the world, producing wheat, barley, soybeans, sunflowers and sugar beets. The region's economic growth depends on agriculture, and agricultural growth depends on a carefully managed water supply. Government studies have projected that the national demand for food and fiber produced in the region will have doubled between 1960 and 2020, putting additional stress on regional water supplies (Hendrickson et al., 1972).

The first task of the research project was to hire a program director to meet and interview community, county and organization representatives throughout the region to determine 1) their interest in water issues, 2) the management roles they presently assume, and 3) the roles they could assume in the future.

After nine months of interviewing a broad spectrum of people and initiating much interest in the topic, the 1983 Red River Valley International Summit Conference was held in Grand Forks, North Dakota. The purpose of the conference was to test the need for a "bottom-up" approach to regional water management and to elicit unity and support from the general public. Two hundred fifty people, many of them mayors and reeves (the Canadian counterpart), gathered to talk about mutual water problems and to determine how local government leaders, water professionals and the public could work together to unify the region over its divisive water issues.

The local CBS evening news program called the conference the most sophisticated effort of its kind in the United States. A resolution was adopted at the conference to develop an association of interests (not a new structure) that would encompass representatives from cities, counties, commissions, states, provinces, private businesses and special interest groups. The conference asked the International Coalition for Land and Water Stewardship, a regional bipartisan group, to accept the responsibility of providing a framework for discussion and resolution of the issues raised at the Summit Conference.

The consensus of the conference seemed to be that the solutions to problems in the Red River Valley lay within a "bottom-up" rather than a "top-down" approach to land and water policy development and implementation. Therefore, asking the federal government to establish a new basin-wide, top-down authority did not appear to be the answer, nor did tampering with the existing legislative management structures which currently set and implement land and water policy within the respective political boundaries.

In July 1984, the Coalition finished its first draft of a model proposal for a Red River Valley policy guidance process structure. It includes a policy steering committee overseeing four specific committees: 1) a structural planning committee to develop and recommend options for an ongoing association of interests; 2) a development committee to encourage public involvement and financial support; 3) an information/education committee to develop a public awareness program; and 4) a conference committee to plan the Second Annual Water Summit Conference. A varied cross section of local officials, business people and public interest groups is represented on the committees to make them truly representative and accountable in the region.

Approval of this policy guidance structure was sought at the Second Annual Summit Conference held in December 1984. Assistance in planning and developing the regional public education programs was sought from the Freshwater Foundation.

This "bottom-up" approach to development and implementation of land and water policy in the region is emerging as a vehicle to unify and involve the various factions within the Red River Basin in the development of a comprehensive land and water policy. It will provide closer communication among the people who formulate water policy, the people who manage water policy and the people who are affected by water policy. By so doing, the region hopes to be better equipped to control flooding, guarantee a water supply during periods of drought, protect water quality and manage soil erosion in the years to come.

Before this effort, there was no significant model for grass-roots, interstate, international watershed-based policy creation and management. Now, the International Coalition for Land and Water Stewardship may offer this divided region a vehicle for long-range planning and cooperation and serve as a prototype for other regions of the country in the coming years.

CENTER FOR THE GREAT LAKES

A second attempt at new organizational efforts to overcome institutional barriers to water management exists in a larger watershed than the Red River Valley. The Great Lakes region, encompassing eight states from Minnesota to New York and the Canadian provinces of Ontario and Quebec, has been the scene of a severe recession in recent years, particularly in the auto and steel industries. Some observers have suggested looking at using the rich natural resources of the Great Lakes to revive the regional economy. Declining federal grants and the demise of the federally funded Great Lakes Basin Commission have further complicated the task of resource management.

In March 1982, the Joyce Foundation of Chicago convened a group of over 100 foundation, corporate, government and conservation leaders, to set an agenda for addressing the region's soil and water problems. One major conclusion of the conference was an agreement that there was a need for a strong and effective private sector institution to 1) address the issues of conflicting resource use, and 2) inform regional decision-makers about the resource problems and opportunities before them. Based on these conclusions, the Center for the Great Lakes was formed.

The Center, a private, nonprofit membership organization, has four major objectives. First, it will identify those resource management issues which are both critical to the Great lakes region and can stimulate a broad base of support there. Second, the Center will provide analyses of these key issues to corporate, foundation and governmental leaders. Thorough dissemination of this information will be a major priority of the Center. Third, the Center for the Great Lakes will encourage informed media coverage of priority regional issues. Fourth, through issue-specific conferences, the Center will seek to encourage constructive discussions of policy alternatives and foster compromises among the diverse interests of the region.

In its first two years of operation, the Center has been involved in publishing a directory of Great Lakes agencies and organizations and a bimonthly regional newsletter in conjunction with the Freshwater Foundation. These publications provide a centralized source of information on current activities in the region. The newsletter also provides a forum for the discussion and analysis of resource issues facing the Great Lakes. The Center has conducted a study of the impact of water on Great Lakes industry and tourism for the Council of Great Lakes Governors. The Center has helped plan symposia for the Great Lakes environmental commissioners and provincial and state legislators to update them on water issues.

The quality and character of life in the Great Lakes communities of Canada and the United States have been shaped by an abundance of natural resources. The preservation of that environment is the overriding aim of the new Center for the Great Lakes. The chief strategy for achieving that goal is to inform and educate a number of important constituencies regarding the Lakes: those with control of financial resources - foundations and corporations; those with access to the public ear - the media; and those with policy-making power - the government leaders.

The Center will not seek to promote the specific interests of any one group. The potential strength of this new organization is its nonpartisan approach and objective perspective on major Great Lakes issues and its ability to provide a central forum for discussion of these issues.

NATIONAL WATER ALLIANCE

The Red River Valley is working on a grass-roots approach to water management. The Center for the Great Lakes was born of institutional efforts midway up the ladder from the grass-roots approach. Still another type of forum has been established to bring together the concerns and resources of federal and local agencies, as well as the business sector and private citizen groups.

United States Congressional leaders have found that state-by-state battles for funding are becoming increasingly bitter. Because of the divisiveness that exists even at the regional level on water issues, several congressmen felt there was a need for an alliance that looked at regional issues from a national perspective. In 1983, five senators and five congressmen from diverse regions created the National Water Alliance.

Many valuable commissions and federal offices have been set up over the years in an attempt to address national water issues. However, there has never been an independent body reflecting public and private involvement but with a direct link to the government - an independent body providing a forum for the development of ongoing national water policies - until the Alliance was formed.

In the past several years, there has been a move in Congress towards the formation of regional coalitions. Senator Dennis DeConcini of Arizona became the co-chairman of one of these groups, the Western Coalition, which addressed natural resources and other problems common to the West. He felt there was a need for a broadened perspective in regard to water issues. After talking with other members of Congress and finding similarities in their home states regarding water problems, Senator DeConcini developed the concept of the National Water Alliance to "avoid any major water crisis or emergency." (Mosher, 1984)

His efforts grew out of concern for federal budget cuts for the Central Arizona Project, a huge pipeline project which would transfer water from the Colorado River to Arizona to provide a future water supply for this fast-growing desert region.

DeConcini conceded that other regions needed federal dollars for water development too, including Eastern states, where aging distribution systems lose hundreds of millions of gallons of water per day to leaky pipes. Historically, more than three-fourths of federal water development funds have gone to Western and Southeastern states (Amdur, 1983).

The idea for the National Water Alliance was tested at a two-day symposium at Philadelphia's Academy of Natural Sciences in 1983. More than 200 water experts, ranging from engineers to educators to government leaders and citizen activists, attended the water symposium. From the speakers and panel discussions, two conclusions were reached: 1) What is needed is a holistic policy framework that considers the entire hydrologic cycle and distinguishes between short- and long-range problems in the assignment of priorities; and 2) information management in all water resource areas needs to be improved.

To that end, the Alliance, through Senator DeConcini, was instrumental in sponsoring an amendment to provide a $600,000 grant for a water resource study in 1984. The funds were used to consider the potential for and define the parameters of developing a national center for water resources research and a national clearinghouse for water resources information. The two-part study was conducted through the Council on Environmental Quality to determine how institutions and individuals in both the private and public sectors could participate in water resource planning and research. In recognition of the joint role of the federal and non-federal sections in this effort, the Alliance independently supported the federal study with a series of symposia around the country.

The Western symposium was held at Scripps Institute of Oceanography in La Jolla, California, to invite public comment and input on the research center and information clearinghouse study.

The Midwest symposium was a public policy forum hosted by the Freshwater Foundation in St. Paul, Minnesota. Its purpose was to consider the roles states and regions could play in the development of a national water policy and to explore the concept of reuse as a specific policy issue.

The Eastern symposium was held in Washington, D.C., to explore the connection between policy trends and how related research can best impact those trends. All three symposia suggested that a key to forward-looking policy and action is multidisciplinary study directed toward institutional, technical, scientific and social concerns.

William Gianelli, Secretary of the Army for Public Works, recently reflected on the changing nature of water management: "The days of letting the federal government sit here in Washington and dictate what ought to be good for South Dakota or wherever are over . . . you have to bring everyone into the process and make them partners. The federal government is just not going to provide it all anymore." (Mosher, 1984)

IMPLICATIONS FOR THE FUTURE

Of these three case studies, one was initiated in a rural area in an effort to unify the region and enable it to effectively manage a resource that is intrastate, interstate and international. One was developed to address conflicting resource use within the Great Lakes Basin, an extensive watershed, half urban and half rural, with varying national and international levels of jurisdiction. The third coalition was developed in response to the need of national policy-makers to overcome regional divisiveness and competition for federal dollars in light of the overwhelming complexity of water issues throughout the United States.

All three coalitions reflect 1) a trend toward integrating rural, regional concerns in watershed management; 2) a private-sector initiative to integrate social, political and economic issues which have hindered effective water management in the past; and 3) cooperation among federal and state agencies, the business and academic community and public interest groups.

In highlighting these organizations, the inference is not being made that these new efforts will be the ultimate answer to managing regional or national water problems. They do, however, reflect important trends toward the development of realistic management strategies for addressing those problems:

These associations see management as needing to integrate quality and quantity, surface and groundwater, structural and non-structural approaches.

They realize that sound management practices cannot be developed without joint cooperation of the public and private sectors.

They are not new bureaucracies. They are associations of interests that cut across bureaucratic, political and geographic boundaries.

Their structures make them more flexible in seeking new financing options and in setting water priorities.

They are based on the premise that water problems will require an increasingly regional, watershed-based approach.

They are willing to admit that the water managers of the future must be "society-wise" as well as "technology-wise."

In the final analysis, these new organizational efforts illustrate that approaches to water management must be as innovative, integrated, comprehensive and fluid as the resource they serve.

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THE ENVIRONMENTAL HEALTH NETWORK: A NEW TOOL FOR PROTECTING PUBLIC HEALTH

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ABSTRACT

Rural and urban health professionals do not necessarily have access to information or an understanding of the complex contaminants which are accumulating in the environment and how these substances mix with other factors such as nutrition, lifestyle and occupation, to increase the possibility of cancer or other environmentally-related illnesses and diseases. A medical doctor or an occupational health nurse may not know that some of the cases of stomach disorders they are treating are sentinel cases of well-water contamination or pesticide usage, unless they are aware of the side effects of such chemicals.

There is an institutional gap between medical/scientific research and those agencies and organizations charged with monitoring, regulation and protection of public health. Research, by its very nature, is a slow process undertaken by specialists in a narrow field of study. They do not report their findings until all evidence is conclusive. When they do comment, it is within highly specialized, technical journals which serve their particular area of study, but are not necessarily reviewed by health professionals and policy-makers.

There is a time lag of a year or more before public health professionals learn of the risk, diagnosis, implications and treatment of new environmental contaminants.

In an effort to address this institutional and time gap, the Freshwater Foundation has established a state prototype for a National Environmental Health Network.

NEWS UPDATES

"In the next few years, the nation will have to address three basic and difficult problems concerning safety of drinking water: carcinogens in source and finished drinking water . . . drinking water in water starved areas . . . and aging distribution systems."

> <u>10th Annual Report of the Council on Environmental Quality</u>,
> Executive Office of the President, Washington, D.C., 1979

"During fiscal year 1980, 28,000 of the 65,000 community water systems in this country showed violations for . . . not meeting drinking water quality standards."

> <u>Report to the Administrator, Environmental Protection Agency</u>
> U.S. Government Accounting Office
> March 3, 1982

Nearly 39 million people - 63% of all rural Americans - drink contaminated and possibly unsafe water, according to a recent report prepared by Cornell University for the U.S. Environmental Protection Agency.

> <u>Science News</u>, July, 1982

"About 50 Lake Elmo residents presented a petition Tuesday to the Washington County Board asking it to provide bottled water to those whose wells contain dangerous chemicals from the Lake Jane landfills."

> <u>St. Paul Dispatch</u>, March 17, 1984

A CHALLENGE TO PUBLIC HEALTH

There was a time when pollution was viewed as an aesthetic issue and pollution control was a luxury. The times have changed and complicated those traditional perceptions of environmental issues. As an advanced, industrial society, we are using more water than ever before in the history of mankind.

With that increased use has come a massive and exotic residue of chemicals and heavy metals released into the environment from agricultural and industrial processes. These substances are released to the environment in many ways, sometimes through smokestacks, landfills or runoff, and find their way into our air and water.

Questions and concern about the impact of these substances on public health led one state medical society, the Minnesota Medical Association and its Committee on Environmental Health, to undertake a study in 1977 and 1978 on drinking water quality in Minnesota. The Committee met with numerous experts from the areas of state and federal government, scientific research, industry and public interest groups.

STUDY RESULTS

Their findings provided a microcosm of the state of the state in regard to water and health. It concluded that large amounts of waste are being disposed of indiscriminately, water is being contaminated, illness is occurring as a result of contamination, available technology is not fully utilized, there is a need for research, and the medical community should assert itself in protecting public health (Minnesota Medical Association Committee on Environmental Health, 1978). The report and the conference on water and health, "Can I Drink the Water?" (cosponsored by the Freshwater Foundation), which grew out of the report, were responsible for changing the direction of Minnesota's State Water Plan.

According to the Official Report from the Minnesota Medical Association on Water and Health released in 1978, "Many chemicals which are used by industry and agriculture find their way into our water supplies and many are getting into our bodies because there is no known practical method to cleanse them from water." (Minnesota Medical Association Committee on Environmental Health, 1978) Many of these synthetic compounds are not filtered by soil or neutralized by bacteria, nor are they filtered by conventional water treatment plants. Water treatment plants were designed to remove bacteria and viruses from water, but not organic chemicals and heavy metals. Yet public health concerns have expanded to include a growing concern for the chronic effects of exposure to these substances. Chlorination of water, introduced in 1907, has been instrumental in killing bacteria and viruses, but is now being found to react with certain chemicals to form chlorinated compounds - many of which are potentially cancerous. Current water treatment technology is becoming obsolete in the face of these new contaminants. These problems are not aesthetic issues; they are public health issues which eventually must be addressed by health professionals.

The Minnesota Medical Association's report recommends that the Association cooperate with the State Department of Health and other appropriate public and private agencies and organizations in the development of a public education program on environmental health. Yet most health professionals themselves either do not understand or are unprepared to deal with the public health effects of such environmental contaminants.

THE INFORMATION GAP

Health professionals do not necessarily have access to information or an understanding of the complex contaminants which are accumulating in the environment, nor do they understand how these substances mix with other factors such as nutrition, lifestyle and occupation to increase the possibility of cancer or other environmentally-related illnesses and diseases. A medical doctor or an occupational health nurse in an urban or rural region may not know that some of the cases of stomach disorders they see are sentinel cases of well-water contamination or pesticide usage, unless they are aware of the side effects of such chemicals.

According to George Waldbott in his book, Health Effects of Environmental Pollutants, "Such minor ailments as intermittent sore throats, dizzy spells, stomach upsets or merely general weakness . . . can be the early signs of a slowly progressive illness caused by air pollutants. The other cardinal feature of airborne illness is not sufficiently appreciated either: that ingestion of contaminated food and water may be of equal if not greater significance than inhalation of contaminated air." (Waldbott, 1978)

According to Dr. Valentine O'Malley, past chairman of the Minnesota Medical Association's Environmental Health Committee and now deputy commissioner of the Minnesota Department of Health, no states in the Midwest region have a procedure for regularly informing and updating medical professionals on the latest research relating to environmental health issues. Budget limitations, time restrictions and heavy workloads preclude this from being a priority.

Meanwhile, state health departments rarely have the funds to randomly test for organics in well water, usually doing such tests only after another agency has suspected a reason to do so. Most of the well-contamination incidents in municipal areas in Minnesota over the last two years were detected after old landfill sites had been mapped by a state agency looking for areas where hazardous waste had been disposed. The state agency informed the health department which then proceeded to document the potential risk to public health after the fact.

Diagnostic tools to establish the effects of pollutants are not readily available to the average physician. Most laboratories that carry out such assays are attached to industrial establishments and scientific institutions to which the practicing physician has no ready access. Although poison centers have been established in recent years in large United States communities, they deal mainly with acute intoxication, not with a chronic, slowly developing disease the cause of which is difficult to pinpoint (Waldbott, 1978).

THE NEED FOR A NETWORK

Over the past seven years the Freshwater Foundation and the Minnesota medical community have collaborated on a series of conferences and professional seminars. During this time, a need has been expressed by the medical community for a network which would foster a greater understanding of environmental health issues, particularly critical water quality issues. This network would involve translating new research findings to such diverse groups as occupational health professionals, county health officers, city engineers, doctors, nurses and government agencies. This would facilitate a broader understanding of the health implications of various pollutants to those charged with monitoring, regulating and maintaining environmental quality and public health.

There is an institutional gap between medical/scientific research and those agencies and organizations charged with monitoring, regulation and protection of public health. Research, by its very nature, is a slow process undertaken by specialists in a narrow field of study. These specialists do not report their findings until all evidence is conclusive. When they do comment, it is within highly specialized, technical journals which serve their particular areas of study but are not necessarily reviewed by health professionals and policy-makers.

There is an estimated time lag of 18 months before public health professionals could learn of the risk, diagnosis, implications and treatment of new environmental contaminants. Also, it is difficult for many health professionals to read the large number of studies, journals and reports in their own areas of specialization.

Dr. Lee Stauffer of the University of Minnesota's School of Public Health has said that while specialists know where to go to get new information in their particular fields, they have no access to any synthesis and analysis of new findings and new trends in a number of specialties. He feels this lack of analysis and synthesis is a critical barrier to helping medical professionals and others understand public health issues more fully. The time lag and lack of synthesis inhibit the effective prevention, diagnosis and treatment of environmentally related illnesses and diseases by medical professionals.

While it is important to the medical community to be continually updated on environmental health issues, it is also important for the public sector to receive accurate and practical information about environmental health concerns. This information would de-emotionalize public reaction to particular environmental problems by stating what is known, not known and what can be done about the particular problem.

If the public could understand the health threats posed by an environmental hazard, the means for controlling it, and the limits of existing data, they would be in a better position to support remedial action (Freudenberg, 1984). County health offices should be fostered as places the public can turn to for such answers to environmental health questions. An informed public can also better assist the medical community in pinpointing sentinel cases of concern.

In a recent article in the American Journal of Public Health, Dr. Nicholas Freudenberg suggests that "as environmental threats to health increase, as the federal commitment to environmental protection diminishes, and as the budgets for these services are strained ever more tightly, environmental health officials will be hard pressed to fulfill their responsibilities for health protection. Well-organized and educated community groups may become valuable allies for those charged with environmental health protection. Environmental scientists, health educators, local health officials, and policy-makers can help these groups to become more effective advocates. . . . Local health authorities might also benefit by establishing ongoing mechanisms for citizens to report environmental problems." (Freudenberg, 1984)

ESTABLISHING A STATE PROTOTYPE

Building on its past efforts in bringing new scientific information, health professionals and the public together to foster a greater understanding of environmental health issues, the Freshwater Foundation is establishing a state prototype environmental health network, in cooperation with the State Department of Health, the Minnesota Medical Association, the Minnesota Public Health Association and others.

The Freshwater Foundation and the medical community feel that this network is something the medical community needs, but it does not have the mechanism to start. The Foundation will act is acting as a catalyst in starting this program.

Developing a prototype health network would involve setting up a regular communication system with both health professionals in the public and private sectors and representatives of key government agencies. This communication process will take the form of:

1. Establishing an advisory board of five state and five nationally known health professionals and scientists who will help select critical issues to be addressed, and provide access to research personnel at medical centers and universities across the country.

2. Hiring researchers, to collect information from these medical centers and universities, and medical writers to analyze and synthesize it.

3. Publishing a bimonthly newsletter for health professionals which will report, analyze and synthesize research findings and their implications to public health in a short, concise, readable fashion.

4. Publishing information brochures on environmental health issues to be distributed to the public in doctors' offices and county health facilities, and short research updates to be published in organizational newsletters.

5. Conducting seminars for regional health officials to be held in different parts of the state over three years' time. Key researchers will be brought in for intensive half-day sessions on specific environmental health issues of regional concern.

6. Improving the exchange of information between the public and the medical community by encouraging the public to address questions and concerns to county health centers. This process will be fostered by the development of public service announcements on environmental health-related topics..

While the Foundation understands that environmental health research is not always conclusive and that researchers are most often reluctant to release new research information before publication, it does feel there is sufficient information that can be released which would make medical professionals aware of specific trends, issues or symptoms to be watched for in a clinical setting. The Foundation has had much experience with investigating and translating research. Since 1974, the Foundation has published the award-winning newsletter, Facets of Freshwater, a newsletter interpreting research being conducted at the University of Minnesota's Gray Freshwater Biological Institute. This publication provides an update on research-in-progress, an interpretation of what this research means to the general public, and its implications for biological management of the environment.

The environmental health network is designed to:

1. Promote a greater understanding of the health implications of environmental pollutants, by relaying new research findings about those substances to the people entrusted with protecting public health.

2. Allow health professionals to pinpoint environmental health concerns before they become serious, putting them in a predictive rather than a reactive

capacity.

3. Serve as a national prototype for health professionals in other states, who will need this type of information network among their own public and private agencies.

4. Enhance rural health care by providing outstate regions with the same information available to urban areas.

5. Provide a centralized process for coordinating and distributing medically accurate information to the public.

6. Provide a basis of knowledge necessary to the development of sound public policy.

7. Develop a mechanism that becomes self-supporting by the medical community through subscriptions, conference fees and publication sales.

MEETING FUTURE NEEDS

The medical community's methods for coping with environmental health issues have not kept pace with either the magnitude or the complexity of air and water contamination. Environmental contaminants have shifted from large quantities of biodegradable substances to substances which accumulate not only in the environment but in the human body as well. These substances carry the threat of synergistic rather than single effects, and they represent a shift from acute diseases to long-range chronic diseases, with symptoms which can take 20 years to become manifest.

Institutional barriers such as time limitations, budget restraints, lack of environmental health training and the variety of medical specialties place medical professionals in a weakened position to provide the guidance and leadership expected of them in addressing environmental health issues. This state prototype environmental health network will serve as a catalyst in preparing the medical community for the unique health issues of the future.

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Paper number 260 Aspect number 2

THE LEARNING IMPLICATIONS OF TECHNOLOGY TRANSFER IN THE WATER SECTOR
AN ANALYSIS OF THE PROBLEM AND SOME PRACTICAL RECOMMENDATIONS FOR ACTION

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ABSTRACT

This paper examines the human factors which influence and constrain the process of technology transfer, with particular reference to transfers which take place in countries with radically different cultures, and to the learning problems which this creates.

Some of the main variables which will be considered are the size and nature of the "technological gap", the size of the "cultural gap", the vocational training systems of the host country and the expatriate organisation(s) which is supplying the knowhow, the methods used for recruiting and training training staff and, finally, the design of an overall learning strategy for the whole project and the evaluation of its effectiveness. In general it will be argued that the cultural values and assumptions made by many Western advisers who work as trainers or technologists in Third World countries act at least as powerfully to constrain the learning experiences of their clients as do the cultural values of the client - if not more so.

PREFACE

This paper applies to the rural water sector in the developing countries of the World some basic ideas about the learning implications of technology transfer which were first put together by one of the authors in a paper presented at the 12th Annual Conference of the International Federation of Training and Development Organisations held in Amsterdam from 15th - 19th August 1983.

The general conclusions recorded there had been reached in the light of the practical experience of the writers and their colleagues working as training consultants based in Britain but carrying out their assignments in both developed and developing countries throughout the world.

Their application in this paper to the rural water sector is based on direct practical experience obtained by the authors in looking at the manpower problems of the water industry in countries as diverse as Barbados, Nigeria, Nepal and Southern India and by other colleagues who have carried out similar assignments in Peru, Ecuador, Tanzania, Sri Lanka and Indonesia.

This overseas experience was itself developed as a result of the ITS being invited in the 1960s to carry out a number of consultancy and teaching assignments in the British Water Industry, ranging from the design of a comprehensive training scheme for water operatives in a large municipal water undertaking to work on general courses for instructors and training officers in the industry. Clients during this period included the British Waterworks Association and the Water Supply Industry Training Board.

One of the authors (John Densham) is an electrical engineer with a background in Work Study who has a particular interest in technical training and in training officer training in the water sector. He collaborated recently with the International Reference Centre for Community Water Supply and Sanitation in the preparation of a training manual on slow sand filtration.

The other author (Eric Greig) is an economist with a special interest in manpower analysis and the quantitive aspects of training in the water sector. He was invited recently to give a paper on the Assessment of National Skilled Manpower needs at a conference convened by the South Asia Region of the World Bank in Sri Lanka (January 1984).

FALSE ASSUMPTIONS ABOUT THE PROBLEM OF LEARNING "ACROSS THE CULTURAL INTERFACE"

It is our experience in observing how technology transfer is managed in practice that the following sequence is followed.

It is assumed that where technology is being transferred - that expatriate staff who manage and implement the transfer process must be technical experts in the relevant field but that they do not need training expertise of any kind.

Having assumed that the transfer is, by definition, a technical task to be carried out on the supply side by people who know the technology and having appointed such people to manage the process, if it goes wrong, it is frequently assumed that the fault must lie in the host country, its people and its culture.

At this point it is usually assumed that if cultural factors are a primary cause of ineffective learning they cannot be changed quickly - and therefore some other solution must be found. This may take a number of forms:-

Expatriate staff are kept in the field to run things on a full time basis whilst they continue to try to teach their own skills to nominated counterparts.

If this arrangement is thought to be too expensive, expatriates from the country which suppled the equipment may be used on an intermittent basis to deal with breakdowns. This may of course, be almost as costly as keeping experts permanently in the field.

Meantime indegenous staff with the appropriate level of general education may be sent to the supplier country for further training in an environment in which similar plant can be seen running "perfectly"---

It hardly seems necessary to stress about the sequence of acting and thinking described above, that the process goes wrong from the start. People who are asked to manage the process of technology transfer at any level, need to know a great deal more about learning design than they do.

Where the transfer process breaks down, it happens at least as often and probably more often <u>because the transfer strategy itself was badly designed</u>, frequently in its learning aspects, rather than because it was adversely affected by cultural factors in the local situation.

Although the expedients described work after a fashion, they are certainly not cost-effective, and postpone the problem rather than deal with it.

AN ALTERNATIVE APPROACH TO THE LEARNING PROBLEMS OF TECHNOLOGY TRANSFER: A CASE EXAMPLE IN THE MANAGEMENT OF CHANGE

As an alternative to the assumptions above, we suggest that technology transfer between organisations from radically different cultures can and should be treated as a problem in the <u>management of change</u>. To deal effectively with it we need to know not only the relevant facts in the situation but we also need to understand something about the process whereby change takes place within and between organisations. For the special purposes of this paper we shall be interested particularly in the learning aspects of the change process - for both individuals and organisations.

In the case of the Water Sector, in the developing countries where most new developments are part of the UN or some bilateral aid programme at least five types of organisations are likely to be involved in the change process ie

The aid sponsoring agency(ies), the consulting engineers, the contractor who in some cases may be from another Third World country, the suppliers of plant or equipment, the client organisation, eg a national water organisation.

The technological gap

In addition to the obvious increase in complexity of operating plant or equipment which the transfer will create, it is necessary to assess such associated changes as likely effects on occupational structure and the demand for skilled labour, likely effects on the skill content of work, the possibility of operating problems becoming easier and maintenance problems becoming harder etc. and finally, possible needs for procedural change eg.in storekeeping, provision of spares etc.

The cultural gap

Although cultural differences are often real constraints on the effectiveness of technology transfer, their effects are greatly exaggerated by people's expectations and by the use of labels and slogans as an alternative to thinking and planning. It is also important to recognise in large projects that cultural difference between contributors on the supply side are at least as big a source of potential difficulty as those between the consortium and the host country

The vocational training and technical education system of the client or host country

The third set of variables which must be considered are the facilities available already in the host country for technical education and vocational training and the extent to which these might be adapted or extended in the short and medium term to meet the needs of the project. An inventory of these facilities and an objective assessment of their quality must be made before the project starts.

Some of the factors to be taken into account in this respect are as follows:-

Plans already in existence and likely to be implemented within the lifetime of the project for increases in the amount of technical education and off-the-job training including its content (eg range of subject matter offered and the level at which subjects are to be taught).

The likely willingness and/or ability of the key institutions concerned to provide tailor-made arrangements to meet the needs of the project.

The flexibility of the local education and training system - measured in terms of such things as its willingness to cooperate with project management.

The above list seeks to illustrate the kind of questions which must be asked in advance of a project involving transfer of technology, so that realistic instead of fantastic assumptions can be made about the contributions likely to be forthcoming from local facilities.

The vocational training resource of the organisation(s) which are managing the change - the need for a manpower plan

Any project involving technology transfer should incorporate a suitable provision for the assessment of likely learning needs and for the management of learning within the project. What is 'suitable' may be quite limited in scope within small projects. In the case of large projects, however, it is essential to set up a manpower unit (however small) within the project - which from the tendering stage onwards can deal with such key tasks as Manpower Analysis and Forecasting, Recruitment, Assessing local training needs, drawing up and implementing a training plan, and finally, Assessing external training resources and choosing them to match and complement the provisions made locally.

The role of trainers in technology transfer

The full time expatriate trainer

Compared with the average run of the mill training jobs in the average established water industry in Western Europe or the USA, the skills which will be particularly required of the expatriate water industry trainer in the technology transfer situation are:

Resourcefulness in putting together learning ideas when the process concerned is so new that all one has as a guide initially are engineering drawings and publicity leaflets.

Skills in group work in working with multi-disciplinary teams of expatriates and local specialists.

The practical skills which are needed to improvise training plans and training aids to make things happen quickly at the sharp end of the project - side by side with the ability to interpret complex knowledge requirements and communicate these to academic institutions in one's own or a foreign country.

Coaching skills of a high order - for use in teaching and training expatriate technologists and both local managers and local trainers how to carry out their roles in training.

Investigative and problem solving skills applied - not just at the planning and early development stages of a project but at every subsequent stage, to ensure that the assessment of plant training needs and the training plan on which it is based is updated on a continuous basis.

Managers with training responsibilities

Expatriate Managers, technologists or supervisors for whom training is a key part of their work can and should be helped to acquire the trainer skills which they need through a combination of short formal instructional modules and planned learning on the job.

Trainer roles within the water sector of the host country

There are three quite different categories of training staff required within the water sector of a single country and their roles in relation to the problem of technology transfer vary greatly.

Training development staff

These are the small number of trainers needed to work at national level as a support service on training to the individual water undertakings throughout their country. The emphasis in their work and training will tend to be on the identifications of training needs, on learning design and on training evaluation.

Training Officers in the individual water undertaking

As the increased amount of technology transfer in the sector grows, the demand for this category of staff increases quite rapidly. Mostly they will be recruited from technical staff already in the sector and their training can conveniently be structured as an integral part of selected new projects. The planning and supervision of their training is a highly appropriate role for training development staff of the kind referred to above.

Instructors in the individual undertaking

As with the training officer we regard the day to day work of the undertaking, particularly its new projects, as the primary vehicle for training instructors - as opposed to the institutionalising of the instructor training process by the establishment of a permanent centre for this purpose.

Further aspects of the change process

In addition to the five groups of activities described above, within any single major technology transfer situation the following provisions need to be made:

The establishment of a steering group to monitor all the learning activities of the project, this should be a small senior management level group including representatives of both project management and host country

The setting up of a number of "project learning groups" - each consisting of an experienced trainer, plus expatriate and local managers or supervisor's - with the responsibility for developing training plans in a selected area of the project.

The manpower section or unit within project management would co-ordinate the work of the various learning groups. The key characteristics of the learning system as described above are its informality and its flexibility. The strongest emphasis is put on the learning processes which take place within the project itself, on the ground, in the host country.

EVALUATING THE EFFECTIVENESS OF A LEARNING STRATEGY FOR TECHNOLOGY TRANSFER

The outcomes of ITS studies and research in the field of evaluation since the mid 1960s stress the following points:-

Most approaches to training evaluation have involved looking backward at a training event of some kind in an attempt to assess its effectiveness.

There is nothing wrong in principle with our trying to do this, but if it is all we do, it can lead to our putting too much emphasis on the training event itself and not enough on the results of having provided the training.

Recent research and development has concentrated on developing concepts and techniques to enable trainers (and managers) to improve their own ability to use "evaluation" as a tool in increasing the effectiveness of their training interventions.

In the approach which ITS now advocates we argue that evaluation activities must look forward into the organisation as well as back into the training event, and in this way tackle head-on the problems of transfer of learning into the work situation ie the application at work of learning which has been acquired during a training event.

In a large project of the kind contemplated in this paper, trainers in focussing attention on the application of skills which have been learned should encourage trainees at all levels, including management trainees (eg local counterparts) who are taking part in off-the-job training events to answer such questions as:-

" What are you learning?

How are you learning it?

How will you apply it back on the job?

How will you develop your learning back on-the-job?

What help will you need and from whom?

How will you know you are being successful in applying your learning?

etc. etc.

This whole approach to the evaluation of learning puts the trainee rather than the trainer at the centre of the process. The trainers role (including managers in their training role) is to monitor the process whereby trainees take responsibility for their own learning - and consider what additional help and encouragement they need.

We suggest that the approach to training and evaluation advocated in the whole of this paper is extremely flexible and capable of being adapted to a wide range of project management styles. We would certainly argue that existing methods of dealing with the manpower aspects of technology transfer leave much to be desired,and that our suggested improvements have the merits of being directed very specifically at those areas of existing practice which seem to need improvement most.

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OPTIMAL MANAGEMENT OF LARGE AQUIFERS FOR IRRIGATION ACTIVITIES

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ABSTRACT

Optimal management of large aquifers used for water supply and irrigation is investigated using unit response functions and an iterative linear programming procedure. The procedure has been applied to a hypothetical aquifer with 150 pumpage unknowns.

Results of the procedure are compared with a global optimization approach to the same problem. Comparison of the two solutions indicates that almost identical solutions can be obtained by the iterative procedure. The iterative procedure, which decomposes the aquifer into several zones, allows the solution of a large number of unknowns at computational demands below that of the global optimum solution.

Keywords: Optimal management, large aquifers, linear systems theory, linear programming, iterative linear programming.

OPTIMAL MANAGEMENT OF LARGE AQUIFERS FOR IRRIGATION ACTIVITIES

Optimal management of large aquifers used for water supply and irrigation is investigated using unit response functions and an iterative linear programming procedure. The procedure has been applied to a hypothetical aquifer with 150 pumpage unknowns.

Results of the procedure are compared with a global optimization approach to the same problem. Comparison of the two solutions indicates that almost identical solutions can be obtained by the iterative procedure. The iterative procedure, which decomposes the aquifer into several zones, allows the solution of a large number of unknowns at computational demands below that of the global optimum solution.

INTRODUCTION

In recent years the extent and magnitude of irrigation activities in the United States and in particular in the areas overlying the High Plains aquifer in the central United States, have forced the managers of the groundwater districts to propose new and severe regulations. Some of these regulations such as the limit on the rate of water table decline are designed to limit irrigation activities. Other regulations are also proposed to promote conservation of this limited resource. Only in the last decade has the optimal management of ground-water resources been the topic of many investigations (see Gorelick, 1983). In formulation of a ground-water or ground-water/surface-water management model, a suitable objective function is first defined. Various physical, legal, and environmental constraints that must be satisfied are subsequently included in the formulation. Depending on the form of the objective function and constraints, linear or nonlinear programming may be used to calculate the optimal management strategy.

For a ground-water aquifer, the objective function may be based on minimization of a total cost function (Maddock, 1972), maximization of total pumpage (Heidari, 1982a, b), maximization of the sum of values of hydraulic head at specified points, minimization of the total water shortage (Willis, 1983), or on other objectives such as the minimization of the sum of the squared difference between demanded and supplied pumpages. The constraints associated with these objective functions may include the maximum allowable drawdown and pumpage, and other quantifiable environmental, physical, and legal constraints.

In a ground-water management model, the response of the aquifer to a management strategy must be incorporated into the constraint or the objective function. This may be achieved by two methods. One method, called the embedding method, incorporates the numerical approximation of the ground-water flow equation directly into the management model as a set of constraints. This method has been used in several studies (see for example Aquado and Remson, 1974; Aquado et al., 1974; Alley et al., 1976). The method is applicable to confined as well as unconfined aquifers (linear or nonlinear ground-water flow equations). Because for every node a different equation must be written, the embedding method also provides the flexibility of considering all nodes in the model as potential well sites. However, in practice, this advantage has proven to be a costly one because for a reasonable size aquifer, the large number of constraints may indeed make this method impractical. Recently Nisai et al. (1983) have proposed the use of this method for the optimal ground-water management of Edward Aquifer in Texas.

Another method which can be used to incorporate the response of the aquifer to stress (pumpage) into the management model is the use of unit response functions (see Maddock, 1972, Morel-Seytoux, 1975, Haimes and Dreizen, 1977). These functions may be calculated for aquifers to which the linear system's theory is applicable (confined aquifers). The existence of these functions for nonlinear systems (unconfined aquifers) has been shown by Maddock (1974b). For drawdowns of less than 25% of the saturated thickness, the difference between the solution of linear and nonlinear equations describing the flow of water through porous media is negligible (Jacob, 1944; Bear, 1979).

While theoretically the unit response functions seem to simplify the structure of the mathematical programming matrix based on the management model, in practice they demonstrate a disadvantage by limiting the size of the management models to a relatively small number of active wells and management periods. The disadvantage of using these functions is due to the computational demands, both in regard to computer storage and time. This disadvantage is particularly a serious one when one applies these functions to large irrigation districts where several thousand irrigation wells may be operational simultaneously. Heidari (1982a, b) had to limit the number of active nodes (pumping wells) to 61, simply because the computer storage and time on a Honeywell 66/60 computer became excessive for larger number of nodes.

In this paper an iterative procedure, which curbs the computational requirements associated with the unit response functions to a great degree, is proposed for the optimal management of large aquifers. The iterative procedure is first described for a generalized management model. It is subsequently applied to a hypothetical aquifer with a linear objective function. To verify the global optimum solution of the iterative procedure, the management model for the hypothetical aquifer is also solved by a noniterative procedure and the results are compared.

THE ITERATIVE PROCEDURE

Consider a large aquifer with several hundred pumping wells. One may write a management model such as

$$\begin{matrix}\text{Max} \\ \text{or} \\ \text{Min}\end{matrix} \quad F = f(Q,s) \tag{1}$$

subject to: $$\underline{Q} \le Q \le \overline{Q}, \text{ and } \underline{s} \le s \le \overline{s} \tag{2a,b}$$

where Q = pumpage, s = drawdown, $\underline{Q}$ = the lower limit of pumpage, $\overline{Q}$ = the upper limit of pumpage, $\underline{s}$ = the lower limit of drawdown, $\overline{s}$ = the upper limit of drawdown, and F is the objective function to be optimized.

For a distributed-parameter transient model, Q and s vary both temporally and spatially. Other constraints may be added to (2). The relationship between s and Q may be expressed by a ground-water flow model such as the two-dimensional flow model for confined aquifers (see Maddock, 1974a). If there are M active wells in an aquifer, the drawdown at a particular well at a particular time may be expressed as

$$s(i,n) = \sum_{j=1}^{M} \delta(i,j,n) \tag{3}$$

where $s(i,n)$ = drawdown at well i at the end of time period n

$\delta(i,j,n)$ = incremental drawdown at well i due to pumpage of well j from the beginning of pumpage to the end of time period n.

Note that in (3), Q is included implicitly through $\delta(i,j,n)$.
If the aquifer is divided into K zones, each with M_k active wells $(M = \sum_{k=1}^{K} M_k)$, then using (3), the drawdown of well i located in zone k at the end of time period n may be written as

$$s(i,k,n) = \sum_{j=1}^{M_k} \delta(i,j,n) + \sum_{\substack{k'=1 \\ k'\neq k}}^{K} \sum_{j=1}^{M_{k'}} \delta(i,j,n) \tag{4}$$

The first term on the right-hand side is contributions of drawdowns from wells in zone k; and the second term represents the contributions of drawdown of all wells in the other zones.

The value of $s(i,k,n)$ is then substituted for the drawdown variable in constraint equation (2b). Clearly, if the second term on the right-hand side of (4) is known, the computational efforts required are reduced. In practice neither of the terms on the right-hand side of (4) are known. One may make an educated guess for the second term on the right-hand side of (4) and write (1) and (2) for each zone k and then solve this reduced problem for the optimal policy for zone k. To calculate an optimal policy for zone $k+1$, the optimal policy for zone k may then be used in conjunction with the educated guesses for the second term on the right-hand side of (4) for other zones except for zone $k+1$ in (1) and (2). Continuing in this manner, one may solve (1) and (2) for K zones. In effect, instead of solving one large mathematical programming problem, one may solve K small problems. The results of one solution for each zone is called one iteration. Iterations may continue until the difference in optimal pumpages from two consecutive iterations is within a pre-specified level of tolerance.

RADIUS OF INFLUENCE

To make the iterative procedure even more efficient, one may estimate an upper limit for the radius of influence of each well R_i. Then, the computation of the right-hand side of (4) may be limited to the wells located within the radius of influence, and (4) may be written as:

$$s(i,k,n) = \sum_{j=1}^{M_{R_{i,k}}} \delta(i,j,n) + \sum_{\substack{k'=1 \\ k' \neq k}}^{K} \sum_{j=1}^{M_{R_{i,k'}}} \delta(i,j,n) \tag{5}$$

where $M_{R_{i,k'}}$ = the number of wells in zone k' located within a radius R_i from well i in zone k.

The value of R_i for each pumping well may be estimated using the empirical formulas proposed by Bear (1979).

Since underestimation of R_i results in errors in the optimal pumpage, one must make sure that overestimation, rather than underestimation, is taking place because modest overestimation produces accurate results at the cost of a small amount of computational efforts. Because of the fact that the technique noticeably increases the potential of the management model for large equations, this cost seems quite tolerable.

DRAWDOWN COMPUTATIONS

The key element in the iterative process described above is the computation of the drawdown at a particular well. Unit response functions seem to be particularly adaptable to this process. Given that the region is divided into K zones and that unit response function as proposed by Maddock (1972) can be applied to the aquifer under study, (5) may be written as

$$s(i,k,n) = \sum_{j=1}^{M_{R_{i,k}}} \sum_{t=1}^{n} \beta(i,k,j,k,n-t+1)\, Q(j,k,t) +$$

$$\sum_{\substack{k'=1 \\ k' \neq k}}^{K} \sum_{j=1}^{M_{R_{i,k'}}} \sum_{t=1}^{n} \beta(i,k,j,k',n-t+1)\, Q(j,k',t) \tag{6}$$

where $\beta(i,k,j,k',n-t+1)$ = drawdown at the ith well in the kth zone after time n due to a unit pulse of pumpage at the j^{th} well in the k'th zone beginning during the t^{th} time period, and

$Q(j,k',t)$ = pumpage of the jth well in the k'th zone during the t^{th} time period.

In (6), values of $\beta(i,k,j,k',n-t+1)$ can be calculated by numerical or analytical solution of the ground-water flow equation. If a set of $Q(j,k',t)$ is assumed, the second term on the right-hand side of (6) becomes a constant. Therefore, the only unknown will be $Q(j,k,t)$, which can be calculated directly through the optimization procedure.

TEST OF THE ITERATAIVE PROCEDURE

To test the computational efficiency and accuracy of the iterative procedure, the hypothetical but realistic aquifer shown in Figure 1 was assumed. This aquifer has four transmissivity zones with values ranging from .01 to .1 ft^2/sec. Steady state conditions were assumed to exist prior to commencement of pumpage. Using the ADI option of the finite-difference model developed by Trescott et al. (1976), the assumed boundary conditions, the four transmissivity zones, and no pumpage, the steady state head was generated as given in Figure 1. To facilitate the computational efforts, the bedrock elevation was assumed at zero throughout the active flow domain. Steady state head values were calculated for 357 active nodes using a closure error of .01 ft.

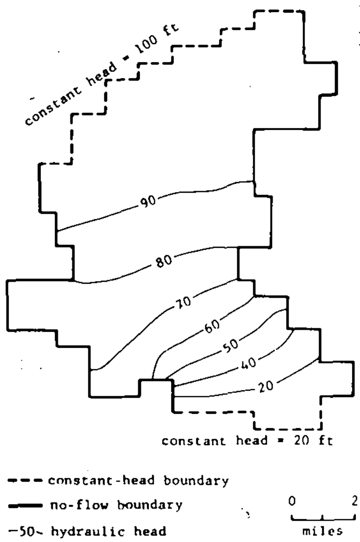

Fig. 1: Steady-State Head.

Drawdown response function coefficients for the iterative optimization were obtained according to the procedure outlined by Heidari (1982a, b) using a head closure of 0.1 ft and a constant storage coefficient of 0.1 in a two-dimensional ADI iterative scheme developed by Maddock (1974a).

DESIGN OF THE MANAGEMENT MODEL

For the hypothetical aquifer, the following simple objective function and constraints were assumed:

$$\max F = \sum_{n=1}^{N} \sum_{i=1}^{M} Q(i,n) \qquad (7)$$

subject to

$$0 \leq Q(i,n) \leq \bar{Q}(i,n), \text{ and } 0 \leq s(i,n) \leq \alpha H(i) \qquad (8a,b)$$

where α is a fraction, and

$H(i)$ is the initial saturated thickness at well i.

In order to check the accuracy and efficiency of the iterative process, the number of unknowns (30 wells and 5 time steps) were chosen so that the solution could be verified by a global optimization management model. The well locations and global indexing system are shown in Figure 2. While a larger number of wells or management periods could have been selected in order to emphasize the utility of iterative optimization for a large-scale system, the identification of 150 unknowns (30 x 5) was believed to be large enough for illustrative purposes during iterative optimization. The moderate dimensionality of the linear-programming problem was also considered to be sufficiently small to facilitate execution of a parallel noniterative optimization. For this example 30 wells (M=30) were placed at nodal locations throughout the aquifer in 10 zones, and five one-year management periods were used to define the planning horizon (N=5). The boundaries of the 10 regions and locations of supply wells in these regions are shown in Figure 3.

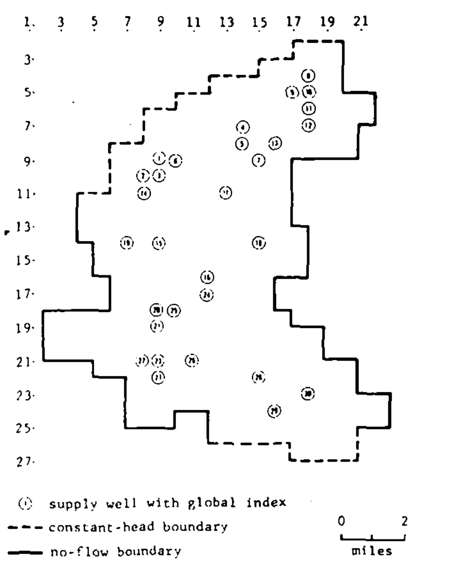

Fig. 2 : Supply Well Locations for Global Optimization.

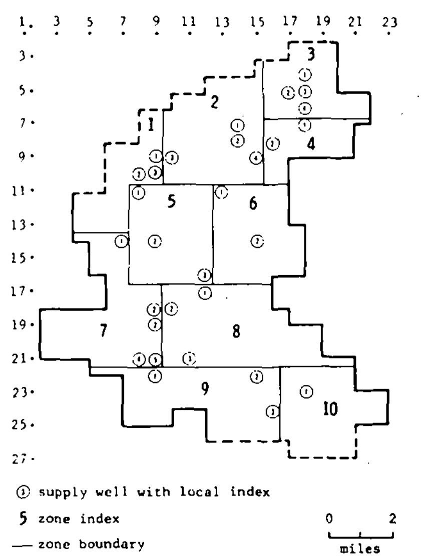

Fig. 3 Zone Boundaries and Location of Supply Wells for Iterative Procedure.

COMPUTATIONAL RESULTS

Global Optimum Solution: In order to check the accuracy of the iterative procedure, a global optimum model was first solved (Fig. 2). The radius of influence of each well was set large enough so that every well was affected by all other wells in the aquifer. The theoretical dimension of the β array was equal to $\beta(30,30,5)$. This solution provides us with the standard with which the iterative solution may be compared. The matrix for this solution contained 151 rows and 150 columns. Using the MINOS version 4.0 linear and nonlinear optimization package developed by Murtagh and Saunders (1977), this model used 181 simplex iterations to converge to the total pumpage of 53.57 cfs. In Table 1 the transient pumpage rates of seven selected wells as obtained by the global optimum are listed.

Table 1. Comparison of transient pumpage rates (cfs) of global optimum with iterative procedure for selected wells.

	Global Solution							Iterative Solution				
Well	Periods						Well	Periods				
#	1	2	3	4	5	Region	#	1	2	3	4	5
1	.5	.5	.5	.5	.5	1	1	.5	.5	.5	.5	.5
5	.5	.5	.441	.329	.289	2	2	.5	.5	.441	.330	.290
10	.5	.330	.211	.174	.157	3	3	.5	.330	.211	.175	.157
15	.5	.5	.5	.5	.5	5	2	.5	.5	.5	.5	.5
20	.5	.5	.367	.254	.186	7	2	.5	.5	.368	.254	.186
25	.5	.345	.152	.098	.069	8	2	.5	.345	.152	.099	.069
30	.193	.091	.078	.069	.062	10	1	.193	.091	.078	.069	.063

Iterative Solution: The iterative procedure was applied to the same aquifer divided into 10 zones as shown in Figure 3. The number of wells in each zone varied from one in zone 10 to five in zone 7. Because of this variation in the number of wells in each zone and the radius of influence of each well, the size of the linear programming matrices for the 10 zones varied from a maximum of 26 rows and 25 columns in zone 7 to a minimum of six rows and five columns in zone 10. This variation in the size of linear programming matrices caused the number of simplex iterations in each zone to vary from 4 to 31. The number of iterations as defined before proved to be somewhat influenced by the initial pumpage rates assigned to each well.

Choice of the Initial Pumpage Rates: In Table 2 the initial pumpage rates, number of zones, total number of wells, number of iterations, and optimum total pumpages for seven runs are listed. Run 1 represents the global optimum as discussed in the previous section. Runs 2 through 7 represent the different experiments performed with the iterative procedure. All runs produced almost exact operating policy (i.e., the individual pumpages agreed to the nearest .001 cfs), and total pumpages as listed in Table 2 are very close to the global optimum (Run #1). The closeness of individual and total pumpages to that of global optimum depends on the error of closure α, or when

$$\left|Q(j,k,t)_i - Q(j,k,t)_{i-1}\right| \leq \alpha \qquad \text{for all } j,\ k, \text{ and } t \qquad (9)$$

the iteration was stopped. In (9), i refers to the iteration number. For runs 2 through 7, α was set equal to .01 cfs.

Table 2. Comparison of results of global optimum solution with iterative procedure.

Run #	Initial Policy cfs/well	# of zones	Total # of wells	# of iterations	Opt. Total pumpage (cfs)	Procedure
1		1	30		53.57	Global
2	0.0	10	30	6	53.59	Iterative
3	0.1	10	30	5	53.59	Iterative
4	0.3	10	30	4	53.59	Iterative
5	0.36	10	30	5	53.58	Iterative
6	0.4	10	30	5	53.58	Iterative
7	0.5	10	30	6	53.58	Iterative

Runs 2 through 7 represent solutions for initial pumpages set at lower and upper bounds (0.0 and 0.5 cfs) of pumpages. These two runs required six iterations each to arrive at the optimal solution. Run 4 with an initial pumpage of 0.3 cfs required the least number of iterations (four).

In Table 1 the individual pumpage rates as obtained by the iterative procedure for the same seven wells as that of the global solution are listed. A comparison of the two tables in Table 1 should be supportive of the convergence capability of the iterative procedure to the global optimum solution.

Based on the data in Table 2, one may conclude that the choice of the initial pumpage rates affects the iteration requirements. However, the variation in iteration requirements due to different initial pumpage rates for this particular example seems small. As is demonstrated by Run 3, the closer the initial pumpage rates to the optimum pumpage rates, the fewer number of iterations is required for the iterative procedure to converege. Since information on temporal and spatial variations of pumpage is not available a priori, and the limited experiments in this study show that different initial pumpage rates affect the computational efforts moderately, one may conclude that initial pumpage rates between the upper and lower bounds are sufficient to get the solution started.

In Figure 4 the rates of convergence to optimal pumpage vs. the number of iterations for iterative runs 2, 4, and 7 are compared. Whereas these runs reproduce the same optimal solution as the global optimum, their initial guesses of pumpage affect their rates of convergence mildly. Runs 4 and 7 are shown to over or underestimate optimal pumpage at the end of iteration 1. This is simply because when the initial pumpage guesses are set at lower or upper bounds, some of the constraints during the first iteration will be satisfied at the extreme or will be violated. During the second iteration when the optimal pumpages of the first iteration are used instead of upper or lower bounds of pumpage, almost no constraint is violated. Therefore, the curves of Figure 4 approach that of global optimum at the end of iteration 2.

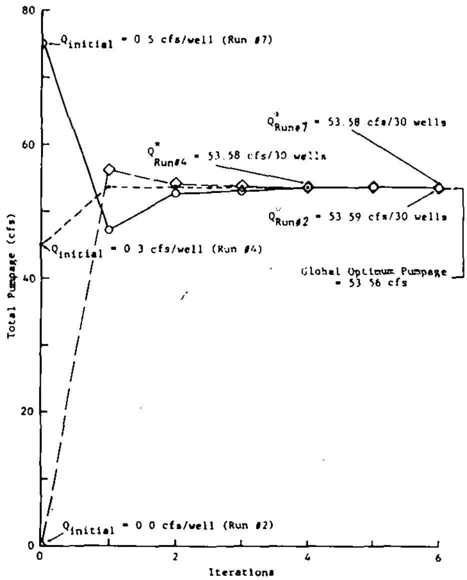

Fig. 4: Comparison of the Convergence Rates of the Iterative Procedure with Different Initial Pumpage Rates to Global Optimum

CONCLUSIONS

In this paper an iterative procedure which can be used for the computation of the optimal management policy of large aquifers with many operating wells has been proposed and tested. Using a hypothetical aquifer, the iterative procedure is shown to converge rather rapidly to an optimal solution which is almost identical to the global optimum solution. The limited experiments performed in this study show that the different initial pumpage rate guesses affect the convergence rate rather insignificantly. When no other information on the initial pumpage rate is available, initial pumpage rate guesses midway between the upper and lower bound of pumpage rates have been shown to be effective in producing an optimal policy.

The major contribution of this study is in its ability to make the optimal analysis of large aquifer systems possible. This is done using zonation of the aquifer together with a radius of influence for each well to decompose the global constraint matrix into several matrices whose sizes may be decided a priori, depending on the computer memory availability. Therefore, no matter what the computer memory, one may divide the aquifer for optimal management policy analysis.

The theoretical computational efforts of the iterative procedure per iteration for one particular decomposition have been shown to be 10 times less than the computational efforts of the global optimum. Whereas the computational efforts per iteration for the iterative procedure can be proved fewer than those for the global optimum solution, the total computational efforts of the iterative procedure depend on the number of iterations used in each case.

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DEVELOPPEMENTS RECENTS DES TECHNIQUES DE TRACEURS APPLIQUEES AUX PROBLEMES DE RESSOURCES EN EAU

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RESUME

Les problèmes de ressources en eau, qu'il s'agisse d'hydrologie, d'hydrogéologie ou de génie chimique, se rapportent généralement à la connaissance des transferts de masse (d'eau ou de substances polluantes). Les méthodes de traceurs et les différentes applications présentées dans cette communication ont essentiellement un but pratique et ont contribué à la solution de nombreux problèmes. De par leur extrême spécificité ces méthodes offrent des moyens d'investigations souvent sans équivalent mais, étant affaire de spécialistes qui ne consacrent probablement pas assez de moyens à les promouvoir ; leurs performances et leurs domaines d'applications restent encore mal connus de la plupart des responsables confrontés aux multiples problèmes de ressources en eau.

La méthodologie générale des traceurs et les exemples d'applications présentés dans ce mémoire ont pour objectif de pallier à cette situation.

Descripteurs : Traceurs, hydrologie, hydrogéologie, caractéristiques hydrodynamiques, transfert de polluants, autoépuration, relations fleuve-nappe, calage de modèles, génie civil, recherche de fuites, débit d'infiltration.

I - INTRODUCTION

La maîtrise de l'utilisation des ressources en eau, tant pour l'alimentation publique ou industrielle que pour l'irrigation ou la production d'énergie impose l'exploitation de réservoirs naturels ou l'aménagement d'ouvrages pour lesquels, sauf circonstances exceptionnelles, se posent des problèmes aussi importants que variés (contrôle et mesures de sauvegarde de la qualité des eaux - choix du point d'implantation des ouvrages - étanchéité des ouvrages existants) et dont l'incidence financière ou les risques encourrus sont rarement négligeables.

Sans prétendre régler tous les problèmes, les méthodes de traceurs paraissent aujourd'hui un outil complémentaire indispensable à la panoplie des techniques classiques utilisées jusqu'alors. Si, malgré leurs développements récents, ces méthodes sont maintenant bien connues des spécialistes qui se consacrent à leur mise en oeuvre et à leur amélioration permanente, il apparait qu'au niveau des concepteurs ou des gestionnaires confrontés à des problèmes de ressources en eau, ces méthodes sont encore relativement méconnues.

La présente communication n'a pas pour ambition de décrire de manière exhaustive les différentes méthodes de traceurs, mais plus modestement de présenter brièvement la méthodologie générale et décrire les possibilités offertes par les traceurs pour l'étude des transferts de l'eau dans le milieu naturel.

Les applications présentées permettent de décrire avec simplicité et rigueur les modalités de transfert de l'eau (ou de substances polluantes) dans les situations les plus complexes. Elles ont essentiellement un but pratique et ont contribué à la résolution de nombreux problèmes concrets. En hydrologie elles concernent principalement les problèmes de transfert de substances polluantes dans le réseau hydrographique et la détermination du pouvoir autoépurateur des cours d'eau.

En hydrogéologie, elles ont trait au transfert de l'eau, vecteur de la pollution, à l'interaction des polluants avec le milieu, et de là au problème général de la sauvegarde de la qualité des eaux et de la détermination des périmètres de protection des captages d'alimentation d'eau publique.

En génie civil elles permettent la recherche et la localisation de fuites dans des retenues ou canaux, l'évaluation des débits réels de fuite et la détermination des modalités de transfert au travers des ouvrages.

II - METHODOLOGIE DES TRACEURS

II.1. Notion de traceurs

On appelle traceur d'une population dont on étudie l'évolution tout élément ayant un comportement identique aux individus de celle-ci et possédant un caractère permettant son identification.

Cet élément peut être naturellement présent dans la population ou ajouté à celle-ci.

II.2. Choix du traceur

La condition essentielle d'utilisation d'un traceur est que celui-ci marque de manière spécifique la population à étudier. Le traçage idéal consiste alors à utiliser un isotope du milieu sous même forme chimique. Cela n'étant que rarement possible, on choisit un traceur de nature différente mais présentant un comportement dynamique, chimique ou physico-chimique identique.

En plus de ces critères, le choix définitif du traceur reste soumis à certaines conditions dont les plus importantes sont :

. toxicité nulle à partir des points de prélèvement

. présence naturelle aussi faible que possible

. détection de grande sensibilité permettant l'utilisation de faibles quantités

. mesure simple, donc peu coûteuse

. faible prix de revient et approvisionnement aisé.

Les traceurs les plus couramment utilisés en hydrologie se répartissent en trois familles :

. les traceurs fluorescents (Rhodamine B, Rhodamine Wt, Eosine ...).

. les traceurs chimiques (Iodure de sodium, Bromure de sodium ...).

. les traceurs radioactifs (Iode 131, Brome 82 ...) ou radioactivables.

De plus, les traceurs naturels du milieu (isotopes stables : Oxygène 18 - Deuterium - Azote 15 - Tritium d'origine thermonucléaire - Carbone 14) peuvent également apporter dans certains cas des solutions aux problèmes posés.

II.3. L'utilisation des traceurs

a/ Marquage

La technologie du marquage dépend à la fois de l'échelle et de la nature des phénomènes à étudier, ainsi que de la structure dans laquelle ces phénomènes interviennent. Le marquage devant être représentatif de la population étudiée, l'injection "idéale" est obtenue par introduction du traceur de telle façon qu'il soit réparti de manière homogène en concentration dans cette population.

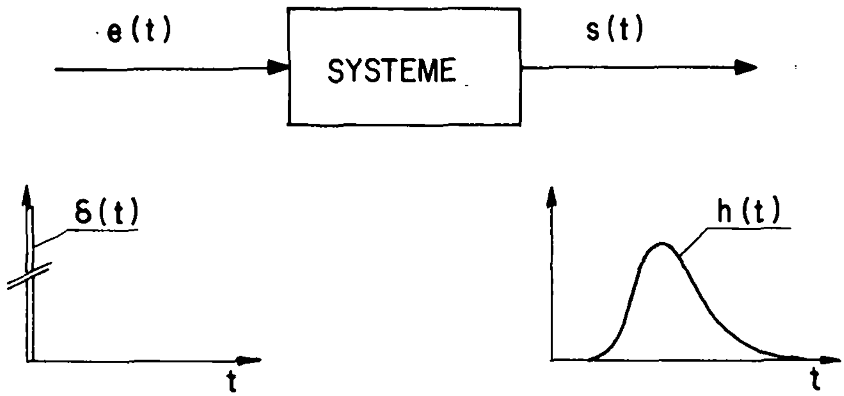
b/ Détection

A chaque traceur correspond une technique spécifique de mesure, celle-ci pouvant aussi bien s'effectuer "in situ" comme c'est le cas pour les traceurs fluorescents ou radioactifs, que par prélèvement d'échantillons sur lesquels on effectue les dosages en laboratoire pour les traceurs chimiques.

II.4. La Distribution des Temps de Séjour

Les méthodes de traceurs dans leur application à l'étude des transferts dans les systèmes consistent à superposer un écoulement de traceur à l'écoulement fluide étudié.

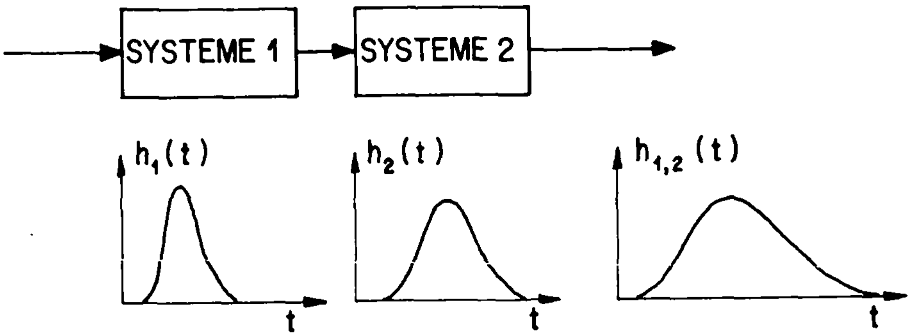
L'information la plus complète que l'on puisse souhaiter obtenir pour caractériser le transfert d'un fluide dans un système est la Distribution des Temps de Séjour (D.T.S.) de ce fluide dans le système considéré. Cette D.T.S. est la distribution fréquentielle de passage, à la sortie du système entre un temps t_0 et $t_0 + dt$ des éléments de fluide entrés dans le système à une date t_0. Pour un régime hydraulique stationnaire, la D.T.S. est un invariant du système considéré.



Soient e(t) et s(t) les concentrations en traceur observées à l'entrée et à la sortie d'un système donné, dont la D.T.S. (h(t)) a été déterminée au cours d'une expérience préalable par injection instantanée de traceur. On montre que ces trois grandeurs ne sont pas indépendantes, mais liées par une intégrale dite de convolution

$$s(t) = \int_0^t e(u)\, h(t-u)\, du$$

Si e(t) est une injection instantanée, type impulsion de Dirac δt, la sortie s(t) est appelée réponse impulsionnelle ou D.T.S. Si e(t) est constant dans le temps (injection échelon), la sortie s(t) est appelée réponse indicielle et est égale à l'intégrale de la réponse impulsionnelle obtenue par injection instantanée de traceur. Cette propriété de convolution est fondamentale dans la méthodologie des traceurs. Ainsi, par exemple, considérons un ensemble de systèmes en série dont on désire connaitre la D.T.S. globale. Pour des raisons techniques, (masse de traceur à mettre en oeuvre, durée expérimentale) il est très souvent plus rentable et tout aussi précis de déterminer les D.T.S. de chacun des systèmes en série $h_1(t)$, $h_2(t)$,... $h_n(t)$ par injection individuelle de traceur. Les opérations de convolution ultérieures permettent alors de prévoir les concentrations à la sortie du deuxième, troisième, $n^{ème}$ système.



De plus, les D.T.S. obtenues peuvent, dans certains cas, être comparées à celles que fournissent des modèles mathématiques représentatifs des phénomènes de transfert dans des systèmes élémentaires. Ces modèles sont variés et pour ne citer que les plus couramment utilisés :

. Modèle piston : le fluide se déplace en tranches parallèles n'échangeant pas de matière entre elles. La D.T.S. est $h(t) = \delta(t-\tau)$; (δ, fonction de Dirac et τ, temps de séjour moyen égal au volume du système divisé par le débit).

. Modèle piston-dispersion : rend compte des échanges de matière entre les tranches parallèles pendant leur transfert dans le système. Ce modèle est caractérisé par un nombre adimensionnel dit nombre de Peclet égal à $P = \frac{uL}{D}$ (u vitesse, L longueur, D coefficient de dispersion axiale).

. Modèle mélangeur parfait : un système est dit mélangeur parfait si le fluide entrant se répartit instantanément et uniformément dans le fluide présent dans le système. La D.T.S. est dans ce cas $h(t) = \frac{1}{\tau} \exp - \frac{t}{\tau}$

Ces modèles simples peuvent être perfectionnés pour mieux représenter la réalité des phénomènes observés. Ainsi dans le modèle piston-dispersion on peut introduire des zones stagnantes échangeant de la matière avec la zone débitante. Le modèle est alors décrit par trois paramètres.

De plus, ces modèles peuvent être associés entre eux en parallèle ou en série pour constituer des modèles composites.

III EXEMPLES D'APPLICATIONS

III.1. Hydrologie

III.1.1. Détermination des paramètres d'écoulement

Soit $h_{1,2}(t)$ la D.T.S. obtenue sur un tronçon de cours d'eau entre deux sections Σ_1 et Σ_2. L'abscisse t du moment d'ordre 1 de cette fonction définit le temps de séjour moyen de l'eau dans ce tronçon, paramètre reliable à la vitesse moyenne de l'écoulement. Ce temps de séjour moyen est égal au volume d'eau compris entre les deux sections Σ_1 et Σ_2 divisé par le débit

$$\bar{t} = \frac{V}{Q}$$

Ce temps moyen peut être obtenu directement par différence des abscisses des moments d'ordre 1 des fonctions concentration-temps enregistrées dans les sections Σ_1 et Σ_2 et résultant d'une injection instantanée de traceur en amont de l'écoulement.

Par ajustement de cette ou ces D.T.S. à un modèle "piston-dispersion" on peut alors déduire le coefficient de dispersion de ce tronçon de cours d'eau, coefficient qui rend compte globalement de tous les phénomènes conduisant à l'élargissement dans le temps d'une injection (d'un rejet) instantané.

III.1.2. Etudes de transfert de polluant

a/ Transfert conservatif

On considère dans ce cas que le polluant est miscible à l'eau et que son transfert dans le réseau hydrographique s'effectue sans perte ou dégradation.

La connaissance de la D.T.S. permet de prévoir, en plus des paramètres temporels et dispersifs précédemment évoqués, les concentrations maximales et moyennes qui seraient observées à l'aval d'une section de rejet.

Si le rejet est effectué de façon instantanée et que l'on connait sa masse, c'est par une simple affinité à la D.T.S. en traceur que l'on déduit les concentrations qui seront observées en aval.

Si le rejet est effectué selon une loi quelconque $e_p(t)$ (en durée et en masse) et si cette loi est connue, c'est par une simple opération de convolution entre cette loi de rejet et la D.T.S. observée en traceur que l'on déterminera les paramètres temporels et les concentrations maximales et moyennes qui seront observées en aval [GUIZERIX J. (1981)].

Cette action prévisionnelle est capitale pour les gestionnaires industriels puisque en effet elle peut être appliquée aussi bien à un rejet accidentel en permettant de définir au mieux les mesures de sauvegarde pour la protection de la qualité des eaux d'alimentation publique, qu'à un rejet contrôlable pour en préciser les modalités optimales n'entrainant pas, par exemple, de dépassement des concentrations maximales admissibles dans les eaux susceptibles d'être captées en aval de la zone de rejet.

Le choix et l'implantation sur un cours d'eau d'une station d'analyse et de contrôle automatique chargée de surveiller des rejets existants ou potentiels doivent être faits en tenant compte de ces informations définissant le transfert de l'eau.

b/ Transfert non conservatif

Le polluant, bien que miscible à l'eau, est soumis, au cours de son transfert dans le réseau hydrographique, à des phénomènes, par exemple de biodégradation.

Il est également possible dans ce cas de déterminer, à titre prévisionnel, en tout point aval, la concentration résultant d'un rejet de ce polluant effectué selon une loi quelconque, mais connue, en amont.

Cette opération s'effectue par un couplage des informations décrivant le transfert de l'eau (vecteur du transport du polluant), obtenues par traceur (substance conservatrice) et des lois de cinétique décrivant la biodégration de la substance dans le milieu [MARGRITA R. (1979)].

Par exemple, dans le cas d'un cours d'eau pollué par des rejets biodégradables, on peut déterminer à titre prévisionnel l'influence d'une modification de la charge polluante (augmentation ou allègement) sur la qualité des eaux. Dans ce cas, la qualité des eaux sera définie par leur teneur en oxygène dissous. Les phénomènes physicochimiques majeurs sont relatifs à la consommation d'oxygène par biodégradation et à la réoxygénation du cours d'eau par aération. On supposera que ces mécanismes sont régis par des cinétiques du premier ordre définis par des coefficients K_1 (biodégradation) et K_2 (réoxygénation) mesurés "in situ". Le couplage des deux types d'information : hydrodynamique et cinétique, réalisés selon la méthode d'analyse des systèmes linéaires, permet ainsi d'accéder aux résultats requis.

III.1.3. Mesures de débits d'effluents industriels

Pour ces mesures sur des eaux très chargées et dans une gamme de pH très étendue, le choix difficile du traceur est généralement résolu par l'utilisation de radioéléments (^{24}Na - ^{82}Br...).

La mise en oeuvre de la méthode de dilution avec, selon le problème à résoudre, l'une ou l'autre de ses variantes (intégration ou injection à débit constant) permet la mesure des débits respectivement de façon périodique ou continue pendant des durées test de 24 ou 48 heures. Un échantillonnage de l'effluent est réalisé en parallèle pour un dosage ultérieur de certains éléments dont on peut alors calculer les flux intégrés pour les mêmes durées.

III.1.4. Etudes de la dispersion en mer

L'implantation d'un rejet en mer, qu'il s'agisse d'un émissaire d'eaux urbaines ou d'effluents industriels, pose en premier lieu le choix de la meilleure implantation possible pour assurer d'une part la dispersion optimale de l'effluent dans le milieu récepteur et éviter sous l'influence des conditions hydrométéorologiques un retour à la côte à des concentrations trop élevées.

Même si les gestionnaires conçoivent un modèle mathématique sophistiqué pour résoudre ce problème à grande échelle, il apparait indispensable de procéder à des expériences de traceurs ne serait-ce que pour caler les paramètres dispersifs à introduire dans le modèle. En effet, malgré les développements modernes sur la structure de la turbulence il n'est pas possible à l'heure actuelle de formuler une expression générale permettant de calculer les tenseurs de diffusivité turbulente. On doit donc faire appel à l'expérience afin de déterminer leur ordre de grandeur, et mieux encore, leur intervalle de variation en fonction des principaux paramètres.

Il n'est pas possible dans le cadre restreint de cette publication de présenter en détail la méthode de traceur mise en oeuvre.

Disons seulement que, résultant d'une injection de traceur, la détection longitudinale, transversale et verticale du panache marqué répétée dans le temps, permet de déterminer l'évolution des coefficients de dispersion longitudinale, transversale et verticale selon la trajectoire du centre de gravité du panache marqué.

III.2. Hydrogéologie

III.2.1. Mise en évidence de communication

L'application la plus simple dans son principe, mais pas nécessairement dans sa réalisation, consiste à mettre en évidence des communications entre deux points. La méthode dite "Tout ou rien" revient à injecter un traceur en un point choisi et à effectuer des dosages au point de collection pour matérialiser cette relation.

Cette méthode est souvent appliquée dans les milieux karstiques pour résoudre des problèmes de ressources en eau, tels que limite de bassin versant, contentieux dans le cas d'alimentation en eau ou des problèmes d'anti-pollution : protection de captage en cas d'implantation industrielle ou urbaine. Cette méthode peut être mise en oeuvre dans les milieux karstiques sur des trajets de plusieurs kilomètres caractérisés par des temps de transit de l'ordre de quelques jours ou mois. Il est fréquent que différents traceurs soient injectés simultanément en des points divers et qu'ils soient restitués en des points multiples. Dans cette opération de marquage dite "multipoint" les traceurs mis en oeuvre sont du type fluorescent (fluorescéine, Rhodamine B, Wt,etc..) ou chimique (Iodure de sodium, Bromure...). Dans ce cas la forme des courbes de restitution du traceur apporte des renseignements sur les modes d'écoulement (court-circuit - mélange avec des masses d'eau importante...) et ces informations sont capitales pour une bonne compréhension des conditions de transit dans le milieu suivant son état hydraulique (étiage-crue). Les isotopes du milieu (deutérium - oxygène 18 - tritium...) peuvent relayer cette méthode de traceur artificiel dans le cas où les transits seraient trop longs ou les dilutions trop importantes. La mesure des concentrations de ces isotopes au cours du temps en différents points peut mettre en évidence des relations ou au contraire infirmer de telles éventualités.

III.2.2. Détermination des caractéristiques des nappes aquifères

Depuis quelques années le développement des modèles mathématiques d'écoulement permet d'effectuer des simulations et de prévoir les écoulements sur de grandes échelles de temps et d'espace. Le réalisme de ces modèles repose sur des mesures de terrain pour leur calage. A côté des mesures classiques (pompages - niveaux piézométriques - etc...) les traceurs apportent des informations majeures pour déterminer les paramètres intervenant dans ces modèles tels que : vitesse naturelle d'écoulement - coefficient de porosité cinématique - coefficient de dispersion - profil des vitesses horizontales.

Les méthodes de traceur permettant d'accéder à ces paramètres peuvent être résumées comme suit :

a/ doublet en écoulement naturel

b/ méthode multipuits avec pompage

c/ dilution ponctuelle

a/ Doublet en écoulement naturel

Deux forages implantés sur une même ligne de courant permettent en écoulement naturel de déterminer deux paramètres essentiels : vitesse naturelle d'écoulement et coefficient de dispersion.

L'opération consiste à injecter un traceur dans le forage amont d'une manière instantanée et à mesurer les concentrations de passage dans le piézomètre aval. Certaines conditions doivent être remplies lors de l'injection, telles que mélange vertical du traceur dans le forage d'injection.

Le traceur utilisé pour ce type d'investigation doit être un bon traceur de l'eau. Des traceurs, tels que Iodure de Sodium sous forme radioactive ou inactive, répondent à ces conditions. Il est par ailleurs possible dans ce genre d'opération d'accéder au coefficient de dispersion transversale par mise en place de forages supplémentaires sur une ligne perpendiculaire à l'écoulement.

La mise en oeuvre de cette méthode nécessite des informations sur la direction exacte d'écoulement de la nappe et du fait de la faible valeur du coefficient de dispersion latérale, il existe des risques de ne pas intercepter la vague de traceur à cause d'un mauvais positionnement des forages par rapport à l'écoulement. Pour pallier cette difficulté, un faible pompage sur le forage aval est souvent pratiqué.

b/ Méthode multipuits avec pompage

Toutefois pour éviter ces inconvénients, la méthode dite "multipuits avec pompage" est souvent préférée à cette dernière. Cette méthode fait appel à un forage central dans lequel est effectué un pompage. Ce forage central est entouré de piézomètres satellites dans lesquels sont injectés des traceurs différents et différentiables dans le forage central de récupération où est effectué le pompage. A partir d'une vague de traceur on peut déterminer pour un couple forage d'injection - forage central ,la porosité cinématique et le coefficient de dispersion. Ces paramètres peuvent être différents pour un autre couple forage d'injection - forage central à cause de la non homogénéité du terrain. Les coefficients de dispersion sont spécifiques d'un certain débit de pompage et leur transposition à des écoulements naturels suppose la validité de l'expression $D = \alpha u$, D coefficient de dispersion, u vitesse moyenne d'écoulement.

Cette méthode est souvent mise en oeuvre car elle conduit par rapport à la méthode précédente à un gain de temps et présente un facteur de sécurité quant à la sûreté d'obtention des résultats.

Les paramètres : porosité cinématique - coefficient de dispersion- sont mesurés sur des couples forage central - forage satellite de quelques dizaines de mètres représentant en général un pas de mesure acceptable quant à sa représentativité. Ces coefficients sont essentiels pour le calage de modèle d'écoulement.

c/ Mesure de profil de vitesses horizontales

L'hétérogénéité verticale des aquifères conduit à introduire dans les modèles une distribution verticale des perméabilités.

La méthode dite de "dilution ponctuelle" permet dans ce cas de déterminer sur une verticale le profil des vitesses horizontales réelles.

Une section de forage crépiné isolée par deux obturateurs gonflables depuis la surface du sol est marquée d'une manière homogène avec un traceur. La vitesse de disparition du traceur dans cette chambre permet alors de déterminer la vitesse de l'écoulement dans cet horizon de terrain compris entre les deux obturateurs.

Cette méthode nécessite l'emploi d'un traceur mesurable "in situ" (fluorescent ou radioactif) et également un taux de crépinage convenable du forage. On peut ainsi explorer par cette méthode le forage sur toute sa hauteur et obtenir le profil des vitesses horizontales.

D'autres méthodes permettent également d'accéder à des informations du même type et de mettre en évidence certaines caractéristiques de terrain telles que : mesures de vitesses verticales dans un forage - méthode du puits unique.

III.2.3. Transfert de polluant

L'ensemble de ces méthodes permettent de prévoir le mouvement d'un polluant dans un aquifère. Ces problèmes prennent des formes très diverses : définition des périmètres de protection des captages, implantation des forages de contrôle de pollution des aquifères, évaluation des risques potentiels de pollution des eaux souterraines dans l'implantation de zones industrielles, d'aires de stockage de déchets... [GAILLARD B. (1976)].

Ces prévisions supposent que le polluant se comporte comme l'eau, fluide vecteur et la situation est définie par des paramètres tels que temps minimal de transfert, concentration maximale et moyenne dans le cas de rejets instantanés ou variables dans le temps, durée d'une contamination....

Toutefois, dans le cas où cette hypothèse ne peut plus être tenue pour vraie, il est alors nécessaire de prendre en compte les mécanismes d'interaction avec le terrain caractérisés par des phénomènes de sorption et désorption. Les relations permettant d'associer ces deux types de phénomènes : hydrodynamique et interaction physico-chimique avec le terrain sont très complexes et présentement seules des interactions du type linéaire et réversible sont modélisables.

Cette modélisation fait appel à des mesures physico-chimiques de sorption et de désorption effectuées en laboratoire sur des échantillons de terrain qui ne sont pas nécessairement représentatifs du milieu naturel. Un effort important est actuellement en cours au niveau de la représentativité des mesures physico-chimiques et des relations permettant de coupler ces deux types de phénomènes : hydrodynamique et interaction physico-chimique pour rendre compte au mieux de la complexité des phénomènes naturels.

III.3. Génie civil

III.3.1. Recherche et localisation de fuites sur retenues naturelles ou artificielles

La méthode consiste à effectuer une injection discrète de traceur à proximité d'une zone suspecte. On définit ensuite l'évolution dans le temps de ce panache sous l'influence des courants sublacustres et, principalement, l'évolution de son interface avec le fond ou les parois de la retenue. Les singularités de cette évolution seront l'indice de zones de fuites dont l'existence et la localisation seront confirmées par le contrôle de l'apparition du traceur à l'exutoire des fuites [GUIZERIX J. (1982)].

Seuls les traceurs détectables "in situ" (fluorescents ou radioactifs) permettent ce type d'investigations à partir d'une embarcation dont la trajectoire est connue en permanence grâce à un dispositif de positionnement implanté sur la rive. Cette méthode a donné des résultats remarquables sur les sites de différents barrages.

III.3.2. Evaluation des débits réels de fuite et détermination des modalités de transfert à travers un ouvrage

Selon le problème posé, le marquage de tout ou partie d'une retenue apporte des résultats très intéressants. Ce marquage de volumes importants s'effectue exclusivement à l'aide de traceurs chimiques ou fluorescents. Les mesures de concentration en traceur dans les émergences permettent après établissement du régime permanent en traceur de préciser les valeurs des débits réels de fuite en les distinguant des apports des eaux météoriques drainées par les appuis ou le massif d'enrochement. De plus, le traitement des données constituées par les courbes de restitution du traceur (fonction de sortie) et l'évolution des concentrations en traceur dans la retenue (fonction d'entrée) fourni, par déconvolution de ces couples de fonctions, les D.T.S. de l'eau dans chacune des circulations, D.T.S. à partir desquelles il est possible de calculer les vitesses moyennes de transfert et les volumes dans lesquels s'effectuent ces circulations.

III.3.3. Mesures de débits d'infiltration

Qu'il s'agisse de déterminer le degré de colmatation du lit d'un cours d'eau dans une zone de réalimentation de nappe ou inversement de localiser des zones de pertes par infiltration par exemple dans des canaux d'irrigation, le problème se ramène à la mesure d'un débit d'infiltration par unité de surface. La méthode de mesure consiste à confiner un traceur dans une enceinte ouverte à la base et plaquée sur le fond du lit. Le traceur qui doit être détectable "in situ" est maintenu homogène en concentration dans le volume de confinement pendant toute la durée de la mesure (mélangeur parfait). Le débit d'infiltration dans l'enceinte de confinement étant compensé automatiquement par apport d'eau extérieure, l'enregistrement de la fonction concentration-temps de disparition du traceur est dans ce cas d'allure exponentielle. L'argument de décroissance de cette fonction permet de déterminer le débit d'infiltration local donc également la vitesse au sens de Darcy et d'en déduire la perméabilité de la couche qui tapisse le fond.

IV - CONCLUSIONS

Cet exposé sommaire des méthodes de traceurs n'est que le reflet des problèmes concrets que notre laboratoire a traité dans la dernière décenie. La sensibilisation croissante aux problèmes de ressources en eau conduit à des investigations de plus en plus complexes. Les différentes approches pour la solution de ces problèmes ne manqueront pas de faire appel aux méthodes de traceurs qui, de par leur développement constant et leur souplesse d'adaptation, se révèleront de plus en plus nécessaires, voire indispensables.

Les recherches actuelles de notre laboratoire pour lesquelles des applications expérimentales sont déjà en cours concernent principalement:

. l'interaction des éléments polluants avec l'environnement

. les relations cours d'eau - aquifère

. la caractérisation des milieux karstiques

. la modélisation des milieux fissurés.

Si, comme nous avons tenté de le montrer, les méthodes de traceurs offrent de par leur extrême spécificité des moyens d'investigation souvent sans équivalent il est toutefois nécessaire de préciser que leur mise en oeuvre (aussi bien lors de la phase d'investigation que d'interprétation) relève d'une systématique rigoureuse dont l'inobservance même partielle peut conduire à des résultats totalement erronés dont l'utilisation ultérieure peut s'avérer très lourde de conséquence. Comme toutes les techniques sophistiquées, les méthodes de traceurs sont affaires de spécialistes. Il est néanmoins bien évident que ceux-ci doivent travailler en étroite collaboration avec les spécialistes des différentes disciplines nécessaires à la solution des problèmes posés.

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PROPOSITION D'EXPLOITATION ET DE GESTION DES AQUIFERES POUR LES BESOINS EN EAU DE LA REGION DE BISSAU

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RESUME

Dans le grand BISSAU, 18 forages exploitent à ce jour la nappe du Maestrichtien et peuvent satisfaire les besoins en eau jusqu'à l'horizon 2000. Etant donné le caractère concentré des forages, leur exploitation risque d'entraîner à terme la perte de l'aquifère par intrusion des eaux salées.

Il est proposé une exploitation de plusieurs aquifères récoltant les eaux sur un front de nappe suffisant, afin de les sauvegarder de toute pollution.

Les différents aquifères potentiels sont décrits et leurs caractéristiques essentielles sont mentionnées.

Pour les trois aquifères majeurs, le bilan à partir des données disponibles est établi. L'infiltration et les ressources sont évaluées.

Un modèle de simulation mathématique est proposé.

CADRE GEOLOGIQUE

La partie occidentale de la GUINEE-BISSAU s'étend sur le bassin sédimentaire mésozoïque "Sénégalais".
Il repose sur le socle paléozoïque qui affleure dans l'Est du pays et forme une structure monoclinale plongeant vers l'Océan à l'Ouest avec une pente de 3 ‰. Il est affecté par un jeu de failles orientées NE et (NNW à NW°.) -(cfr. fig. 1 et 2).

POTENTIALITES AQUIFERES

1. LES NAPPES DES TERRAINS PLIO-QUATERNAIRES

Les terrains plio-quaternaires forment une couverture sur la totalité du bassin sédimentaire.
Ils sont de nature sableuse et argileuse et renferment plusieurs aquifères superposés.
Les aquifères supérieurs ont un caractère discontinu ; ils sont de faible épaisseur (2 à 3 m.) et de faible extension.
Les aquifères inférieurs s'étendent sur la majeure partie du bassin sédimentaire ; ils ont entre 2 et 12 m. d'épaisseur.
L'alimentation est directe et le drainage a lieu au niveau des vallées, consistant en de nombreuses petites sources ou lignes d'émergence diffuse.
L'aire d'alimentation est grande et les potentialités aquifères sont indéniablement importantes. Par contre, les aquifères présentent l'inconvénient d'être hétérogènes en épaisseur et perméabilité. Leur exploitation intensive n'est pas réalisable. L'exploitation de ces aquifères est à réserver aux besoins de l'hydraulique villageoise.

2. LE COMPLEXE AQUIFERE DU MIOCENE

Le complexe aquifère est représenté dans la partie du bassin sédimentaire à l'Ouest de la ligne Nord-Sud passant par BISSAU.

L'aquifère a 2 à 10 m. d'épaisseur à l'Est de son extension et sa puissance augmente vers l'Ouest.
Il est constitué de bancs de calcaire, de grès calcaire et de calcaire marneux.
Le toit du complexe se trouve sous la couverture plio-quaternaire et le mur sur les sables et argiles de l'Oligocène. Les aquifères sont captifs.

L'alimentation de la nappe est limitée à la zone de sub-affleurement des calcaires sous les sables plio-quaternaires.
On ne connait pas exactement le drainage de la nappe, mais on suppose qu'elle s'écoule dans la mer formant un interface eau douce/eau salée au droit de BUBAQUE et dans les nappes supérieures.
Les prélèvements par captage sont peu importants.

Si les aquifères du complexe miocène ont des eaux de bonne qualité et peuvent fournir des débits intéressants, ils n'ont, en regard des besoins pour l'alimentation en eau de BISSAU, que des réserves et une alimentation limitées. L'invasion de l'aquifère par les eaux salées déjà observée à hauteur de BUBAQUE deviendrait importante à la suite d'une exploitation intensive de cet aquifère. Compte tenu de ces éléments et des autres disponibilités aquifères, il ne peut pas convenir pour l'alimentation en eau de BISSAU.

3. NAPPE AQUIFERE DE L'OLIGOCENE

La nappe aquifère de l'Oligocène est trouvée dans la partie septentrionale du bassin sédimentaire, à l'Ouest du méridien passant à FARIM.
Plus au Sud, elle apparaît sous le Plio-Quaternaire en une mince bande à la limite du Miocène. A partir de sa limite orientale, la nappe va en s'épaississant vers l'Ouest.

L'aquifère oligocène est situé dans une série de nature essentiellement sableuse avec quelques bancs à lignite, pyrite et argile.

On a une ou plusieurs séquences sableuses constituées de sable grossier à la base et devenant progressivement plus fin vers le sommet.
Dans la partie supérieure de la formation, on observe des variations latérales de faciès.

L'aquifère Oligocène repose en discordance sur la surface altérée des marnes et calcaires de l'Eocène et dans la partie orientale de l'extension de l'aquifère oligocène sur le Maestrichtien.

La nappe de l'Oligocène est captive sur l'ensemble de son extension.

Le volume de l'aquifère et l'étendue de la zone d'alimentation sont suffisants pour envisager son exploitation.

4. COMPLEXE AQUIFERE DU PALEOCENE-EOCENE (P_{1-2})

Le complexe aquifère du Paléocène-Eocène occupe la majeure partie du bassin sédimentaire sénégalais et on le retrouve sur toute la partie située à l'Ouest du méridien de MANSABA.

Le complexe se compose d'une alternance de grès-calcaires, calaires et marnes.

Rarement, on trouve des sables qui représentent une altération des grès ; le calcaire est plus ou moins fissuré et karstique.

L'allure de la surface piézométrique est variable entre la cote + 3 et -23.

L'alimentation des nappes de l'Eocène se produit par infiltration à travers le Plio-Quaternaire à l'extrême NE de leur extension en GUINEE-BISSAU et probablement aussi au SENEGAL.
Elles sont probablement alimentées aussi par la nappe du Maestrichtien, à la base, en plusieurs endroits.
Les pertes de ces nappes semblent avoir lieu surtout par décharge dans les nappes de l'Oligocène et du Miocène.

L'alimentation et le volume de la nappe sont importants ; l'aquifère est exploitable.

5. COMPLEXE AQUIFERE DU MAESTRICHTIEN (K_{2m})

Le complexe aquifère du Maestrichtien s'étend sur l'ensemble du bassin sédimentaire occidental.
Dans les grandes lignes, il est limité à l'Est par le Rio GEBAL jusqu'à l'estuaire du CORUBAL. Dans la partie méridionale du pays, la limite orientale correspond au méridien de 15°.

A l'Est, on trouve des sables quartzeux lités, à grain fin avec passées grossières. Des intercalations d'argile noire et du grès en "lentilles" sont fréquentes.
Vers l'Ouest, ce faciès passe latéralement et progressivement à une série de grès, grès calcaires et calcaires avec rares intercalations d'argilites.
Au Sud, on trouve à la base de cette série un faciès lagunaire avec lignite, grès argileux, argilites.

Le toit du complexe aquifère est recouvert à l'Ouest en concordance par les calcaires, marnes et grès paléocènes, à l'Est le Maestrichtien érodé et le toit sont en discordance, soit avec les argiles sableuses et argiles de l'Oligocène, soit avec les sables Plio-Quaternaires.

Le mur de la formation est constitué par la surface érodée du Paléozoïque à l'Est et du Crétacé inférieur à l'Ouest.

Le niveau de la nappe varie entre -20 et +3.

La zone d'alimentation du complexe Maestrichtien se trouve en bordure Est du bassin.
Les pertes s'effectuent par décharge dans les autres nappes superposées et par déversement dans la mer.
Les nappes du Maestrichtien représentent la ressource potentielle aquifère souterraine la plus importante du pays.

CARACTERISTIQUES ESSENTIELLES DES AQUIFERES

1. CARACTERISTIQUES HYDRODYNAMIQUES

	Débit spécifique l/sec/m	Transmissivité m^2/sec.
Plio-quaternaire	0,05 - 1,5	10^{-4} - 10^{-3}
Complexe Miocène	0,01 - 0,8	-
Oligocène	0,01 - 9	10^{-4} - 6.10^{-3}
Paléo-éocène	0,01 - 25	10^{-4} - 2.10^{-3}
Maestrichtien	1 - 5	$7,5.10^{-3}$ - 2.10^{-2}

2. CARACTERISTIQUES CHIMIQUES

	Minér.globale g/l	pH	Fe^{++} mg/l	
Plio-quaternaire	0,02 - 2	4 - 7	0,1 - 3	20 mg/l
Complexe Miocène	0,2 - 0,7	6,8 - 8	0,1 - 0,5	eau dure 20 mg/l Si
Oligocène	0,1 - 1,5	5 - 8	jusqu'à 3mg/l	eau dure, sodique
Paléo-éocène		6,6-8,8	1 mg/l	eau dure
Maestrichtien	0,2 - 3,5		0,1 - 1	27 mg/l Si, douce

BILAN DES NAPPES

Les différentes nappes semblent s'écouler vers le NWW.
Les nappes du Maestrichtien sous l'effet de l'exploitation à BISSAU ont un infléchissement au Nord de BISSAU, vers le Sud.
Jusqu'en 1981, on peut considérer que l'exploitation des aquifères était négligeable.
Le bilan d'eau jusqu'en 1980 est exprimé par :

$$P = Er + R + I$$

où : P = précipitations

Er = évapotranspiration déterminée selon la méthode de Thornwaite et avec un stock d'humidité équivalent de 100 mm.

R = ruissellement

I = infiltration

A partir des données hydrologiques et de l'analyse des hydrogrammes de crues, on a pu établir pour les zones d'alimentation la loi générale suivante :

. l'écoulement de base $Q_b = \frac{14,5}{100} (I + R)$,

. l'écoulement temporaire $Q_t = \frac{51,6}{100} (I + r)$

. l'écoulement superficiel $R = \frac{33,9}{100} (I + R)$

Pour le Maestrichtien et l'ensemble des deux aquifères : Oligocène et complexe Paléocène-Eocène, on peut considérer que l'infiltration est limitée au flot de base Qb. A défaut de prélèvement, ce volume d'eau recharge par l'intermédiaire de failles subverticales les aquifères sus-jacents ou s'écoule dans la mer.

Q_t correspond essentiellement à l'alimentation du Plio-quaternaire.

1. Bilan du Maestrichtien

Nous considérons un front de nappe de 100 km. de longueur.
L'aire d'alimentation vaut 3920 km^2.

Pour la période 1967 - 1981, on a en moyenne :

	an (mm)	%	m^3 annuel (3920 km^2)
P	1 483	100	58,1 10^8
E_r	776	52,3	30,4 10^8
R	240	16,2	9,4 10^8
Q_b	102	6,9	4,0 10^8
Q_t	365	24,6	14,3 10^8

2. Bilan de l'ensemble Oligocène - Paléo - Eocène

Nous considérons un front de nappe de 60 km. de longueur. L'aire d'alimentation vaut 3600 km^2.

Pour la période de 1967 - 1981, on a :

	an (mm)	%	m^3 annuel (3920 km^2)
P	1 596	100	57,5 10^8
E_2	755	47,3	27,2 10^8
R	285	17,9	10,3 10^8
Q_b	122	7,6	4,4 10^8
Q_t	434	27,2	15,6 10^8

RESERVE, TAUX ET DUREE DE RENOUVELLEMENT

	Réserve	Taux de renouvellement	Durée de renouvellement
Maestrichtien	2,7 $10^{11} m^3$	1,5 10^{-3}	675 ans
Oligocène-Paléo-éocène	6,7 $10^{11} m^3$	6,3 10^{-3}	160 ans

INTRUSION D'EAU SALEE

Les eaux souterraines continentales se déversent dans les océans, tandis que de même les eaux salées s'infiltrent dans le sol sous-marin et s'écoulent vers les continents. Etant donné la différence de densité, un équilibre s'établit et le contact des eaux douces et salées forme une interface inclinée vers l'intérieur du continent.

Les débits s'écoulant dans la mer sont faibles et négligeables devant l'infiltration. La possibilité de salination au contact de dôme de sel est à retenir.

BESOINS EN EAU DE LA REGION DE BISSAU

Ils ont été évalués (en l/sec.)

	BISSAU-DISTRIBUTION	IRRIGATION	TOTAL
1985	144	50	194
1990	190	100	290
1995	257	150	407
2000	343	200	543
2005	454	250	704
2010	603	300	903

DISPONIBILITES

Dans le grand BISSAU, 18 forages exploitent à ce jour la nappe du Maestrichtien. Les potentialités technologiques des puits à pleine utilisation valent 660 l/sec. et devraient couvrir les besoins jusqu'en 2000. Les forages ont été réalisés en 1980-81 et leur durée de vie moyenne est estimée à 20 ans.

Il faut souligner que l'exploitation de 660 l/sec. sur un front de 5 km., soit 132 l/sec/km. excède la réalimentation évaluée à 126,8 l/sec./km sur laquelle doit être préservée une certaine quantité pour éviter l'invasion de l'aquifère par les eaux salées.

SCHEMA D'EXPLOITATION A L'HORIZON 2010

De l'analyse des schémas-types possibles à l'horizon 2010, il ressort que le schéma optimal est l'exploitation conjointe du Maestrichtien et de l'ensemble Oligocène-Paléo-Eocène.

L'exploitation du complexe Oligocène-Paléo-Eocène nécessiterait, soit un traitement des eaux, ou une ligne d'exploitation en amont de BISSAU. De manière plus économique, il y a lieu de consacrer l'exploitation du complexe Oligocène-Paléo-Eocène à l'irrigation et le Maestrichtien à la distribution en eau de BISSAU.

L'exploitation des aquifères resterait en deça de l'infiltration afin de limiter l'intrusion des eaux salées. Le Maestrichtien peut fournir 100 l/sec./km de front de nappe (on a réservé 20 % de l'infiltration afin de se prémunir contre l'invasation des eaux salées). Les forages du Maestrichtien se doivent d'être répartis sur 5,9 km. de front de nappe, orientés selon un axe NNE/SSW. La capacité technologique des puits devrait être de l'ordre de 45 l/sec. et 14 forages seront nécessaires, les forages réalisés en 1980-1981 étant mis hors service.

L'Oligocène-Paléo-Eocène sera exploité pour l'irrigation, la capacité technologique des puits est de l'ordre de 20 l/sec. et l'infiltration est de 186 l/sec./km de front de nappe.

Afin de minimiser les interactions entre puits, les puits du Maestrichtien (300 m. de profondeur) doivent être espacés de 600 m., le Paléo-Eocène (200 m. de profondeur) de 580 m. et dans l'Oligocène (60 m. de profondeur) de 200 m.

PRECAUTIONS - EXPLOITATION OPTIMALE

L'optimisation tant du point de vue purement hydrogéologique qu'économique ne peut se faire qu'à partir d'un modèle de simulation mathématique. Le but du modèle est d'étudier, non seulement la possibilité effective d'exploiter le débit requis sans abaissement prohibitif des niveaux, mais surtout l'incidence de la progression possible de l'eau salée vers les futurs captages en fonction de leur site d'implantation.

Il sera nécessaire de rendre compte de la complexité des différentes nappes superposées et leurs effets interactifs.
La zone à modéliser devra être suffisamment grande et correspondre à un rectangle de 180 km. sur 150 km. dans lequel il faudra prévoir environ un millier de mailles dont le côté varierait entre 1 et 10 km. Dans chaque maille, seront calculées la piézométrie et la salinité ; il sera en effet important de pouvoir représenter les variations de salinité d'une autre façon que sous la forme simplifiée d'un interface.

Le modèle prendra par conséquent en compte deux types d'équation :

- les équations relatives au bilan d'eau,

- les équations relatives au bilan de la masse de sel.

Pour différents scénarios correspondant à différents modes d'exploitation on pourra représenter l'évolution du niveau piézométrique et de la salinité. On pourra simuler en cas de surexploitation de la nappe les effets de réalimentation locale pour barrer l'intrusion de l'eau salée.
Mais il est primordial de pouvoir effectuer un calage du modèle en prenant des levés piézométriques et de salinité relevés.

A cette fin, il est essentiel de procéder aussitôt que possible à des campagnes de tels levés avec forages de piézomètres. Il est aussi essentiel de pouvoir estimer au mieux l'infiltration réelle dans les nappes concernées, et dans ce but il est indispensable d'installer des stations de jaugeage sur les rios existant dans la zone d'alimentation des nappes.

Nous suggérons le programme minimal suivant :

(1) - station de jaugeage sur le Rio MANSOA près de MANSOA et sur le Rio MANSABANDIM près de BISSORA pour l'aire d'alimentation de l'Oligocène et du Paléo-Eocène.
Pour chaque station de jaugeage, on définira l'aire des bassins versants correspondants,

- relevé pluviométrique à MANSOA, BAMBADINCA et BISSORA.

(2) - relevé piézométrique sur un nombre maximal de forages atteignant les aquifères concernés dans l'aire située entre le littoral Atlantique et le méridien passant par MANSABA, relevé conductivimétrique sur les mêmes forages.

(3) - forage de piézomètres et notamment dans la zone délimitée par QUINHAMEL, SAFIM et NHACRA, selon une densité de 1 piézomètre par maille carrée de 5 km. de côté, soit au minimum 16 piézomètres, chaque piézomètre étant en fait un ensemble de trois piézomètres, un pour le Maestrichtien, un pour l'Oligocène et le troisième pour le Paléo-Eocène.

L'ensemble de ces installations préliminaires devrait être réalisé à très court terme. Cinq années de mesures devraient suffire pour caler le modèle mathématique.

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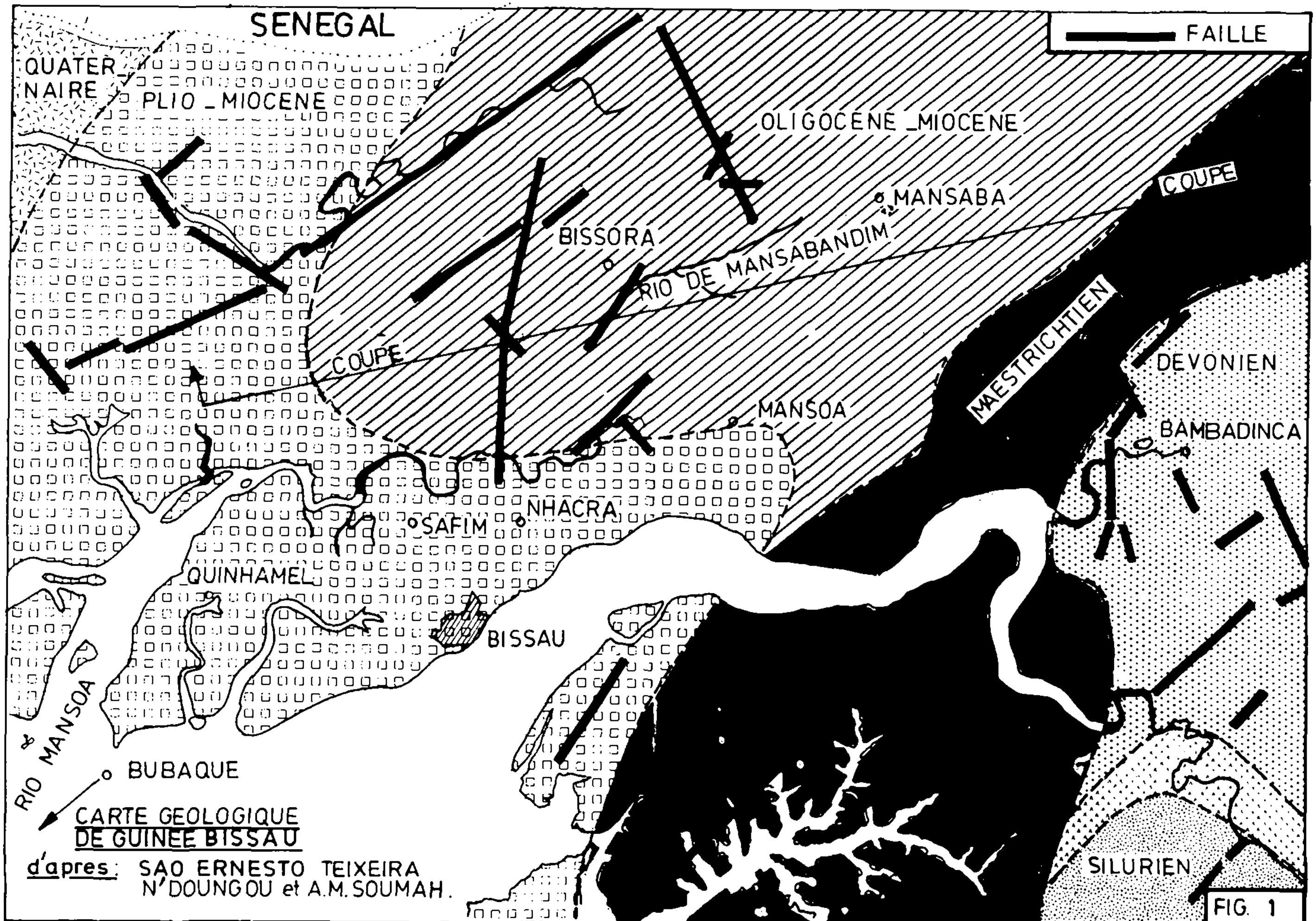
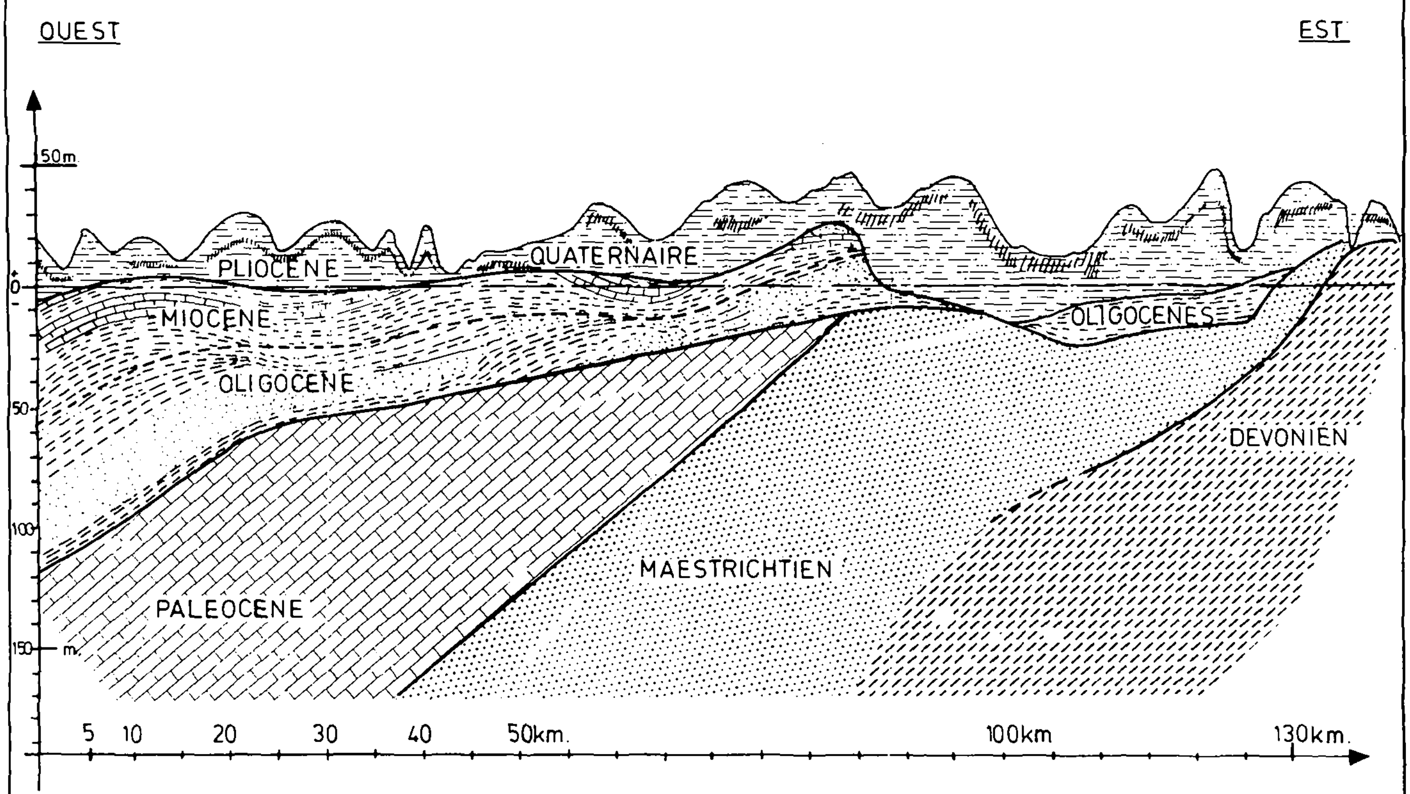


CARTE GEOLOGIQUE
DE GUINEE BISSAU

d'apres: SAO ERNESTO TEIXEIRA
N'DOUNGOU et A.M. SOUMAH.

FIG. 1



COUPE GEOLOGIQUE d'apres : PETERS et STEPANISTCHEV.

FIG. 2

IWRA Vth WORLD CONGRESS ON WATER RESOURCES
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SCHEMA D'EXPLOITATION POUR L'ALIMENTATION EN EAU DE LA REGION DE DJIBOUTI

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RESUME

A partir des données disponibles, on établit au mieux les potentialités aquifères de la région de DJIBOUTI. A cette fin, on se réfère largement aux données établies par la mission de la coopération hydrogéologique allemande (CHA). En fonction de l'évaluation des besoins, une stratégie d'exploitation des aquifères est établie. Celle-ci est dominée par les cycles hydrologiques secs propres au SAHEL et par la prévention contre la pollution de l'aquifère par les eaux océaniques. Une meilleure définition des aquifères et des termes du bilan hydrogéologique est préconisée ainsi que la gestion des aquifères par modèle mathématique.

MOTS CLES

BASALTE, BILAN, CYCLE HYDROLOGIQUE SEC, DJIBOUTI, INFILTRATION, INVASION DES EAUX SALEES, SUREXPLOITATION.

ENVIRONNEMENT GEOLOGIQUE

La structure géologique est illustrée par les figures 1 et 2.
La région de DJIBOUTI est constituée d'un soubassement de formation volcaniques composées de tufs volcaniques acides (rhyolites de l'Oligocène supérieur, de coulées de rhyolites potassiques (Miocène moyen) et l'alternances de laves basiques et acides du Miocène.
Les coulées de rhyolites ont quelques dizaines de mètres d'épaisseur.
Dans la série mixte du Miocène, les coulées basiques sont peu épaisses (quelques mètres) et souvent profondément altérées en argile; les coulées acides sont formées de trachytes de plusieurs dizaines de mètres d'épaisseur.

Au-dessus, s'étalent les coulées basaltiques Pliocènes (B 1) formées en général d'un empilement régulier de coulées épaisses de 1 à 3 m. séparées par des niveaux scoriacés de quelques dizaines de mètres mais la surface occupée par cette formation en affleurement est de plusieurs centaines de km2, couvrant une partie importante de la région. Ces basaltes subissent l'altération en boule.

Sur ces basaltes Pliocènes reposent les coulées de basalte de couverture Plio-Pleistocène (B 2). La puissance totale de cette série d'extension très importante est variable et peut atteindre plusieurs dizaines de mètres. En surface, ces coulées sont localement recouvertes par une couche d'argile d'altération rouge. Quelques petits dômes basaltiques poinçonnent ces formations suivant des alignements correspondant à des jeux de fractures.

La série B 2 est recouverte par des formations sédimentaires Pléistocènes à actuelles formées de conglomérats (Pléistocène ancien) et de dépôts fluviomarins ou éoliens récents (argile sableuse alluvionnaire, blocs et galets des cônes de déjection, dépôts coquilliers, reg).

Au voisinage de la côte, les formations récifales se sont bien développées sur les basaltes sous-jacents (B 1) tectonisés.

L'épaisseurs des dépôts récents est variable mais souvent inférieure à 10 m.

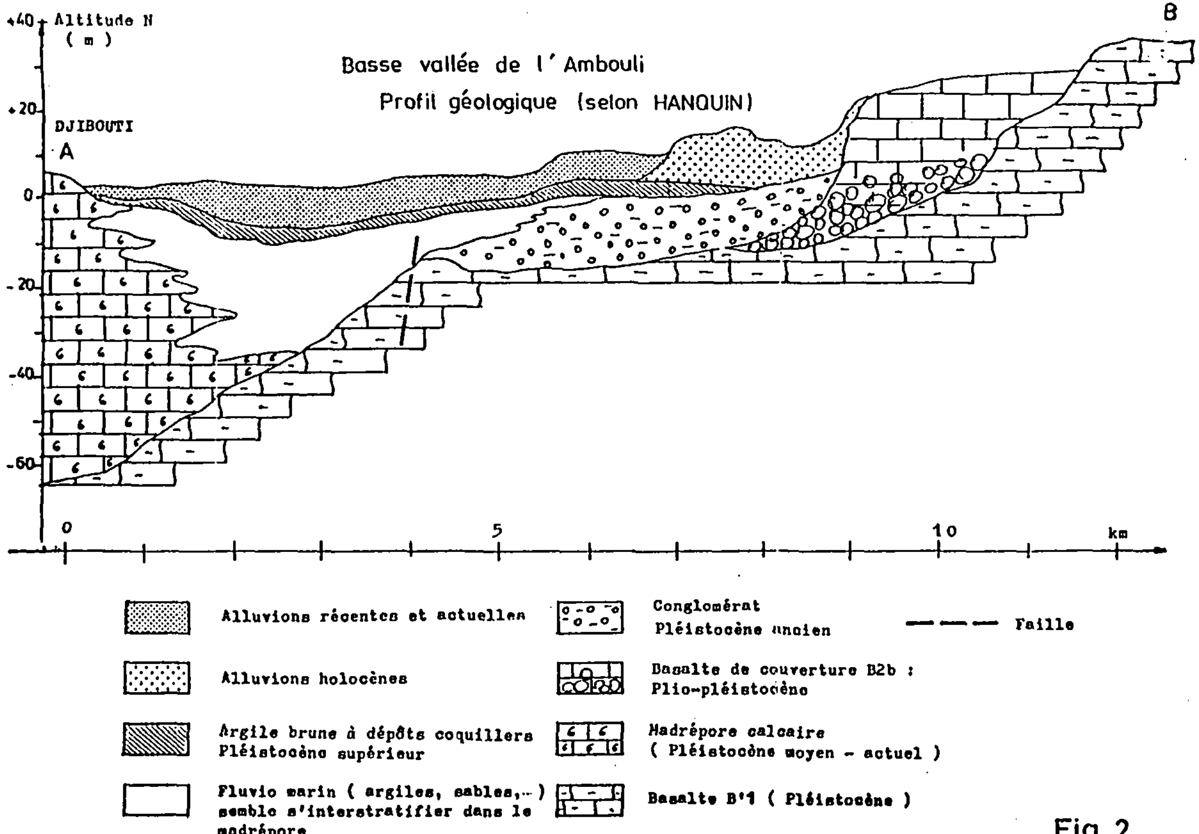
La tectonique cassante a engendré des fractures de direction N 130 à N 150, E - W liées à l'effondrement du Golfe de la côte Est.

CADRE HYDROGEOLOGIQUE

Les principaux aquifères possibles sont les conglomérats du Pléistocène ancien.

La nappe contenue dans les conglomérats est une nappe locale caractérisée à sa base par la présence d'eau salée limitant considérablement son exploitation. Par contre, la nappe des séries basaltiques présente une grande continuité.

Au droit de l'Oued AMBOULI, les levés piézométriques montrent trois axes de drainage dirigés vers le Nord-Est.
Les gradients de la nappe à ces endroits varient entre 1 $^{\circ}/_{\circ\circ}$ dans les basaltes à 1,5 $^{\circ}/_{\circ\circ}$ dans les conglomérats.
Pour l'ensemble de la nappe continue de la région de DJIBOUTI, le gradient moyen vaut 8 $^{\circ}/_{\circ\circ}$.



Altitude N
(m)
+40
+20
0
-20
-40
-60
Basse vallée de l'Ambouli
Profil géologique (selon HANQUIN)
DJIBOUTI
A
B
0
5
10
km
Alluvions récentes et actuelles
Alluvions holocènes
Argile brune à dépôts coquillers Pléistocène supérieur
Fluvio marin (argiles, sables,..) semble s'interstratifier dans le madrépore
Conglomérat Pléistocène ancien
Basalte de couverture B2b : Plio-pléistocène
Madrépore calcaire (Pléistocène moyen - actuel)
Basalte B'1 (Pléistocène)
Faille

Fig. 2.

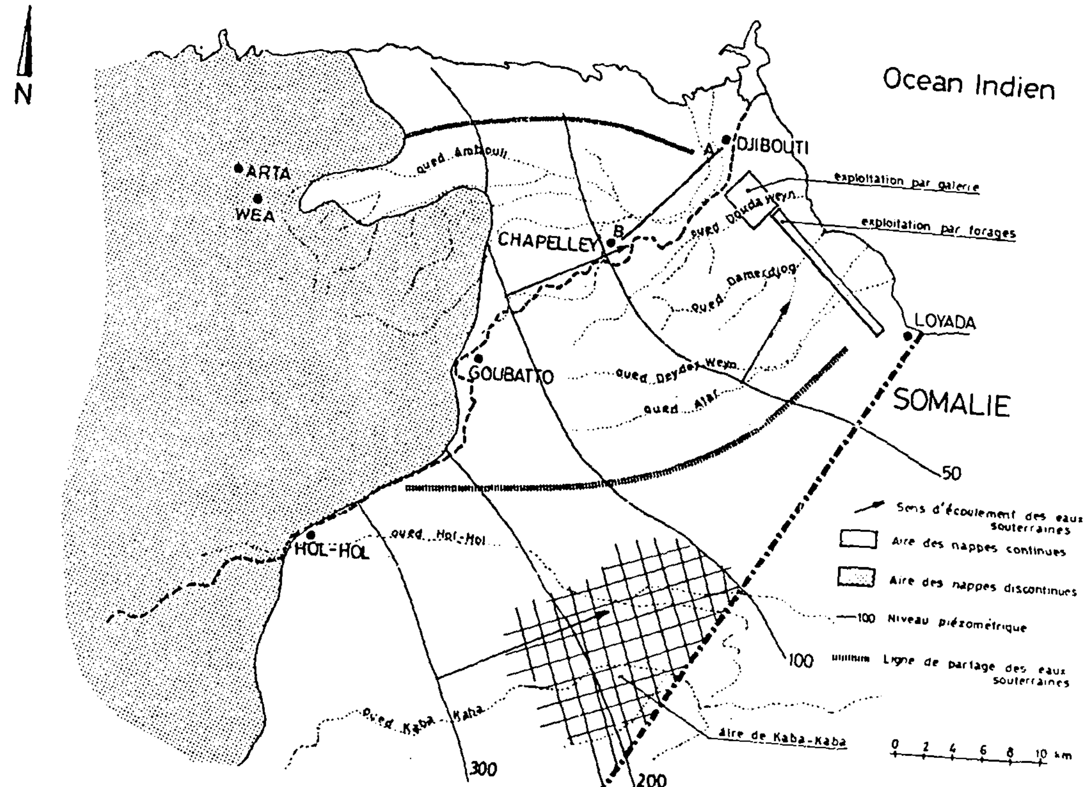

N
Ocean Indien
DJIBOUTI
A
ARTA
WEA
oued Ambouli
exploitation par galerie
exploitation par forages
CHAPELLEY
B
oued Douda Weyn
oued Damerdjog
LOYADA
GOUBATTO
oued Deydey Weyn
oued Alaf
SOMALIE
50
HOL-HOL
oued Hol-Hol
oued Kaba-Kaba
aire de Kaba-Kaba
300
200
100
Sens d'écoulement des eaux souterraines
Aire des nappes continues
Aire des nappes discontinues
100 Niveau piézométrique
Ligne de partage des eaux souterraines
0 2 4 6 8 10 km

Fig.1

Pour les basaltes récents de la région de DJIBOUTI, la transmissivité varie entre $1,2.10^{-4}$ et $2,6.10^{-3}$ m2/sec.
La transmissivité globale est évaluée à 3.10^{-3} à 4.10^{-3} m2/sec.

BILAN DES NAPPES

1. DEFINITION

Le bilan hydrogéologique global a pour but d'établir la balance entre les apports et les pertes, d'étudier ainsi la variation des réserves et donc de déterminer les ressources exploitables.
Sur de nombreuses années, dans les conditions naturelles et sans interventions humaines, il y a conservation des réserves conformément au cycle de l'eau.

La période considérée est l'année hydrologique moyenne (moyenne sur 23 années).

Les apports d'eau sont les suivants :
Précipitation (P); Apports des autres unités hydrologiques (Qa*);
Infiltration due à l'irrigation (Q (u)*).

Les dépenses sont :
Evapotranspiration réelle (Er); Ecoulement ou ruissellement (Q); Prélèvement d'eau (Qex); Pertes vers les autres unités hydrologiques (Qp).

La variation des réserves s'écrit : $\pm$ dW.
Pour de longues périodes hydrologiques égales à plusieurs années hydrologiques, le terme dW est considéré comme négligeable.

Le bilan s'écrit alors :
$P + Qa + Q(u) = E_r + Q + Qex + Qp.$

* Ces termes sont négligeables dans les cas étudiés ci-après car l'irrigation est de faible importance et l'infiltration en résultant est négligeable vis-à-vis des autres termes. Les apports des autres unités hydrologiques le sont aussi, étant donné la continuité de la nappe dans cette vaste région.

2. DEFINITION DU BASSIN HYDROGEOLOGIQUE

A part un pointement de basalte miocène (basalte du DALHA), immédiatement à l'Ouest de DJIBOUTI et une bande côtière d'environ 5 km. de large recouverte de formations sédimentaires peu épaisses, toute la région SW de DJIBOUTI jusque HOL - HOL est caractérisée par la série stratoïde sub-affleurante.

La fracturation intense de cette série peu altérée contribue le plus souvent à faire de cette série un aquifère particulièrement intéressant.

De l'étendue de cette série résulte une nappe continue qui s'étend depuis une ligne HOL - HOL - ARTA à l'Ouest jusque la côte de l'Est.

Le bassin hydrogéologique est limité au Nord par la ligne de partage des eaux souterraines, à l'Ouest par la ligne de crête partageant les eaux s'écoulant vers la mer et vers les plaines intérieures et au Sud par la limite Nord du bassin versant de l'Oued WAHAYYI ou HOL - HOL.

L'exutoire de la nappe continue à l'Est est limité entre DJIBOUTI et LOYADA. La surface du bassin ainsi définie est de 1 032 km2, en sommant la surface des bassins versants : AMBOULI (588 km2), DEYDEY WEYN (319 km2), DOUDA WEYN (65 km2), DAMERDJOG (60 km2).

3. EVALUATION DES TERMES DU BILAN

La précipitation annuelle moyenne sur les différents bassins versants constitutifs vaut :
AMBOULI : 188 mm/an, DEYDEY WEYN : 184 mm/an, DOUDA WEYN : 156 mm/an, DAMERDJOG : 162 mm/an.

La précipitation annuelle moyenne (1959 - 1982) sur tous les bassins d'alimentation est de 183 mm/an.

L'évaporation réelle annuelle moyenne vaut (méthode de PENMAN) :
AMBOULI : 84 % de précipitation annuelle moyenne = 158 mm/an,
DEYDEY WEYN : 82 % de précipitation annuelle moyenne = 151 mm/an,
DOUDA WEYN : 82 % de précipitation annuelle moyenne = 133 mm/an.

L'évapotranspiration réelle annuelle moyenne sur tout le bassin est de 152 mm/an = 83 % de P.

Eaux écoulées : Le coefficient de ruissellement annuel est défini comme étant le rapport entre le volume écoulé du bassin versant pendant un an et le volume des précipitations annuelles du même bassin. Il a été établi pour les bassins de l'Oued AMBOULI et vaut 0,06.

Pour les autres oueds appartenant au bassin considéré, c'est-à-dire DEYDEY WEYN et DAMERDJOG, il n'existe aucune mesure d'écoulement.

Le coefficient de ruissellement dépend essentiellement de trois facteurs : la végétation, la pente et la nature du sol. Ces facteurs étant très semblables pour tous les bassins versants considérés (absence de végétation, pente de l'ordre de 1 %, reg recouvrant les basaltes), il est donc acceptable , en l'absence d'informations plus précises sur les écoulements, de prendre sur l'ensemble des différents bassins versants un coefficient de ruissellement annuel moyen de 6 % de la précipitation annuelle moyenne.

Dans ce cas, Q = 0,06 x 183 = 11 mm/an.

Eaux infiltrées : L'évaluation des quantités d'eau infiltrée dans le bassin hydrogéologique de la région de DJIBOUTI peut être approchée de quatre manières distinctes et indépendantes.

1ère approche : L'infiltration moyenne correspond au débit moyen de l'eau s'écoulant vers la mer et dont le volume est approché par la relation de Darcy :
Q = T.i.L.; T = transmissivité : 3.10^{-3} à 4.10^{-3} m^2/sec.; i = gradient;
L = front de nappe : 18 km.; Q : 14 à 18.10^6 m^3/an.

2ème approche : L'infiltration résulte uniquement de l'infiltration dans les Oueds au cours des crues. Celle-ci dépend de plusieurs paramètres comme le nombre de jours avec écoulement, les caractéristiques du sol à travers lequel l'eau s'infiltre, etc... Pour l'Oued AMBOULI, le nombre moyen de jours de crues par an ainsi que l'infiltration moyenne par jour de crue dans le lit de l'Oued (0,25 m/j.) a été déterminée par différence de

jaugeage entre stations situées à l'amont et à l'aval (OUEAH et DJIBOUTI). En tenant compte des lits mineurs et de l'estimation des surfaces inondables, on considère le minimum d'infiltration correspondant au lit mineur et le maximum à la surface inondable. Il est pris ensuite un tolérance de 50 % à cause de l'incertitude des données de base.
On extrapole ensuite pour les autres Oueds du bassin hydrogéologique de la région de DJIBOUTI.

On a :
Infiltration Oued AMBOULI
. lit mineur = 1,8 km^2 : 0,5 x 0,25 m/j. x 1,8 km^2 = 4.10^6 m^3/an.
. surface
inondable = 4,1 km^2 : 0,5 x 0,25 m/j. x 17 j. x 4,1 km^2 = $9.10^6 m^3$/an.
Autres Oueds
. lit mineur = 2,5 km^2 : 0,5 x 0,25 m/j. x 17 j. x 2,5 km^2 = $6.10^6 m^3$/an.
. surface
inondable = 6,2 km^2 : 0,5 x 0,25 m/j. x 17 j. x 6,2 km^2 = $13.10^6 m^3$/an.
TOTAL : 10 à 22.10^6 m^3/an.

3ème approche : En définissant l'infiltration comme étant le solde des précipitations (P) et de la somme de l'évapotranspiration réelle (Er) et l'écoulement par les oueds (Q), on a pour le bassin versant de l'Oued AMBOULI (588 km^2) :

P = 188 mm : $110,5.10^6$ m^3/an; Er = 158 mm : 93.10^6 m^3/an; Q = 11 mm: $6.5.10^6$ m^3/an; Q = (Er + R) = 19 mm, soit 11.10^6 m^3/an.

Par extrapolation du bassin versant de l'Oued AMBOULI (588 km2) au bassin hydrogéologique de DJIBOUTI (1 032 km^2), on a : $19,3.10^6$ m^3/an.

4ème approche : L'infiltration résulte d'une part de l'infiltration à travers les sédiments des zones inondées des Oueds, tel que décrit dans la seconde approche, dont on retiendra la valeur moyenne ($9,1.10^6$/m^3/an) et d'autre part de l'infiltration à travers les basaltes altérés des bassins versants constituant l'ensemble du bassin hydrogéologique de la nappe de DJIBOUTI.

Le deuxième terme (infiltration sur l'ensemble du bassin versant) a été évalué d'après des modèles d'infiltration établis à DJIBOUTI aérodrome, LOYADA et ARTA. Ces modèles reposent sur l'hypothèse d'une capacité limite supérieure d'infiltration de 75 mm. par jour. Si les précipitations journalières sont supérieures à cette valeur, le solde résulte en écoulements. La partie des précipitations journalières inférieures à 75 mm. est prise comme infiltration après avoir au préalable déduit la lame d'eau équivalente pour satisfaire l'évapotranspiration potentielle et éventuellement réalimenter le stock d'humidité pris ici égal à 100 mm., valeur généralement admise pour l'établissement des bilans.
Les modèles donnent une infiltration de 3 % des précipitations pour DJIBOUTI-Aérodrome, 6 % pour LOYADA et 6 % pour ARTA, soit 5 % des précipitations annuelles moyennes. Ces valeurs établies pour le bassin versant de l'Oued AMBOULI sont extrapolées pour l'ensemble du bassin versant de la nappe.

I bassin = 0,05 x $110,5.10^6$ x $\frac{1032}{588}$ = $9,7.10^6 m^3$/an
I oued = $9,1.10^6$ m^3/an
I total = $18,8.10^6$ m^3/an.

Des 4 approches, on a :

1. (Darcy)	=	14 à 18.10^6 m^3/an
2. I Oued	=	10 à 22.10^6 m^3/an
3. P - RQ - Er	=	$19,3.10^6$ m^3/an
4. I bassin + I Oued	=	$18,8.10^6$ m^3/an

La valeur moyenne de l'infiltration vaut : $17,4.10^6$ m^3/an (= 17 mm).
Le volume d'eau infiltrée s'écoule vers la mer d'une part et est exploité d'autre part.

Volume d'eau exploité : La nappe est exploitée par galeries et forages (cf. fig.1).
Le volume d'eau exploité pour l'alimentation de DJIBOUTI s'élève à 9.10^6 m^3/an (1982), soit une lame d'eau équivalente de 9 mm.
Les prévisions s'élèvent à : $15,8.10^6$ m^3/an en 1992 et $19,3.10^6$ m^3/an en 2002.
Le volume d'eau susceptible d'être utilisé pour l'irrigation de 110 ha. 25 km. à l'W. de DJIBOUTI est estimé à 1.10^6 m^3/an, soit 1 mm.
Qex = 9 + 1 = 10 mm.

Bilan
$I = Q_p + Q_{ex}$ et $Q_p = 17 - 10 = 7$ mm.
L'équation du bilan devient : $P = Er + Q + Q_p + Q_{ex}$ soit pour un an
183 - 152 + 11 + 7 + 10 mm.,
soit : 100 = 83 + 6 + 4 + 5 = 98 %, le bilan ferme à 2 % près.

VOLUME D'EAU EXPLOITABLE DE LA NAPPE DE DJIBOUTI

Limitation du volume d'eau disponible - Risque d'invasion d'eau salée.

Il y a lieu de tenir compte du risque d'invasion saline à l'approche du littoral. Ce risque d'invasion saline qui, le cas échéant, polluerait de manière irréversible une partie de l'aquifère entraîne une limitation des débits exploitables.

Par application de la loi de GHYBEN-HERZBERG, on peut évaluer la longueur du biseau d'eau salée à une centaine de mètres quand il n'y a pas exploitation de la nappe. En exploitation moyenne de la nappe (9.10^6 m^3/an) celui-ci atteindrait 200 m. En considérant que le biseau d'eau salée ne peut se rapprocher de plus de 3,0 km de la ligne de puits développéa parallèlement à la côte, le débit annuel moyen s'écoulant vers la mer est estimé à $3,7.10^6$ m^3/an.
Le solde exploitable vaut (17,4 - 3,7) 10^6 m^3/an. : $13,7.10^6$ m^3/an.

Débit maximal exploitable en cycle hydrologique sec.

Malheureusement, l'analyse statistique des pluviométries montre que pour les 23 dernières années, à trois reprises pour des cycles de 3 ans (1964 - 1966, 1973 - 1975, 1974 - 1976), les précipitations annuelles moyennes ont été de 62 % des précipitations moyennes de la période de 23 ans. A défaut de mieux et en première approximation, on considère que l'infiltration est proportionnelle aux précipitations.

Les eaux infiltrées qui y correspondent valent :

$$q = 0,62 \times 17,4.10^6 = 10,8.10^6 \ m^3/an.$$

De plus, il y a lieu de garder un débit minimal s'écoulant vers la mer.

Moyennant certaines mesures conservatoires qui seront explicitées plus loin, il faudrait contenir le biseau d'eau salée à 1,5 km. de la ligne de puits. La longueur du biseau d'eau salée correspondant vaut : 3,4 - 1,5 = 1,9 km., le débit correspondant s'écoulant vers la mer est de $0,8.10^6$ m^3/an Cette valeur 10.10^6 m^3/an correspond à 57 % de la valeur moyenne exploitable, valeur très comparable à la valeur de la règle de bonne pratique qui consiste à prendre 50 à 60 % du débit moyen disponible à défaut d'une meilleure connaissance des paramètres de l'aquifère et de l'exploitation d'un modèle de simulation en cas de risque de pénétration des eaux salées.

A titre de comparaison, on estime qu'en général en milieu perméable avec un gradient de 4 m/km., soit un rabattement de 3,4 x 4 = 13,6 m. (possible en cycle hydrolique sec), la pénétration du biseau d'eau salée se produit à la vitesse de 500 m. par an. (Das Gupta e.a. 1979).

REMARQUE

La durée du cycle sec, correspondant au tarissement de la nappe a été prise arbitrairement égale à trois ans. Il va de soi que cette valeur doit être précisée. A cette fin, un réseau de piézomètres doit être implanté; leur suivi régulier (mensuel) permet d'établir avec plus de précision le gradient de la nappe et son évolution. En fonction du gradient, on établira les débits de la nappe s'écoulant vers la mer ainsi que les fluctuations du débit.

L'hydrogramme de la nappe est établi à partir des variations du débit en fonction du temps, à partir duquel on établit le coefficient de tarissement de la nappe. A partir de cette valeur, il est possible d'estimer au mieux la durée de la période critique sans ou avec peu d'infiltration.

Indice de pénétration d'eau salée

Le risque évoqué ci-avant de pénétration de l'eau salée dans l'aquifère est confirmé par les observations dans 3 forages : E9, RG4 et RG5, où les débits spécifiques sont faibles (1 à 10 m^3/h/m) et où les teneurs en chlorure sont élevées et varient entre 1 090 et 1800 mg./l.*

Pour ces forages, les débits d'exploitation de l'ordre de 20 à 36 m^3/h. avec de faibles débits spécifiques ont entraîné des rabattements importants. Ainsi, par exemple, pour 30 m^3/h. avec 2 m^3/h/m, on a $\frac{30}{2}$ = 15 m de rabattement.
La cote du niveau statique (niveau de l'eau avant exploitation) est de 1 m. environ.
L'exploitation emmène le niveau de l'aquifère à 15 - 1 = 14 m. sous le niveau moyen des marées. Le cône de rabattement, ainsi crée correspond à une surexploitation locale de l'aquifère et a entraîné une pénétration locale et une remontée des eaux salées.

Stratégie à adopter pour l'alimentation en eau de DJIBOUTI

Les besoins en eau pour DJIBOUTI ont été évalués à : $15,8.10^6$ m^3/an en 1992 et $19,3.10^6$ m^3/an en 2002. Il est dès lors nécessaire de recourir à l'exploitation d'un autre aquifère.

* On admet généralement que le front d'eau salée est caractérisé par une eau à 2 000 mg/l. de chlorure.

L'exploitation de l'aquifère de l'aire de KABA KABA est suggérée. Une estimation de l'infiltration (Rapport CHA) a permis d'évaluer des potentialités à 10 à 15.10^6 m^3/an. Il va de soi que les différents termes de l'évaluation de l'infiltration doivent être précisés afin de contrôler et confirmer ces valeurs.

1. lère phase (1983 - 1987) - Exploitation de la nappe de DJIBOUTI

Le volume potentiel moyen est de $13,7.10^6$ m^3/an (434 l/sec.).
Les ressources exploitables de la nappe de DJIBOUTI en cycle hydrologique moyen suffisent aux besoins évalués à 13.10^6 m^3/an (1987) et à augmenter de 0,5 à $1,0.10^6$ m^3/an pour l'irrigation prévue à l'Ouest de DJIBOUTI. En cycle sec (probabilité de 10 à 15 %, arbitrairement 3 ans à moins de 60 % des précipitations annuelles moyennes), le volume maximum devra être réduit à 10.10^6 m^3/an correspondant à une réduction de 29 % de besoins estimés, par des mesures autoritaires momentanées.

- Augmentation de la capacité technologique d'exploitation de la nappe de DJIBOUTI.
  Elle est évaluée (1982) à $9,4.10^6$ m^3/an. par la réalisation de forages supplémentaires vers le Sud, et par le dédoublement des conduites, on peut l'amener à $14,9.10^6$ m^3/an.

2. 2ème phase (1987 - 1992)

La deuxième phase consistera essentiellement en la mise en application des résultats des mesures recueillies au cours de la première phase. Le modèle mathématique sera ajusté et "calé" à l'aide de ces mesures et rendu pleinement opérationnel.

A partir des résultats des reconnaissances géophysiques, des forages et essais d'eau réalisés sur l'aquifère de KABA KABA et des résultats des simulations sur modèle mathématique, les travaux d'infrastructure pour l'exploitation de cet aquifère seront entamés. A la fin de la deuxième phase, l'aquifère de KABA KABA devrait pouvoir fournir potentiellement $5,10^6$ m^3/an. à DJIBOUTI.

La valeur du volume des potentialités aquifères de DJIBOUTI sera précisée par le modèle mathématique. Les travaux d'extension des captages seront décidés et optimisés en fonction de la valeur des potentialités prédites par le modèle.

En 1992, les besoins de DJIBOUTI sont estimés à $15,8.10^6$ m^3/an.
Volume qu'il y a lieu d'augmenter de $1,10^6$ m^3/an requis pour l'irrigation et industries situées à l'Ouest de DJIBOUTI. Le total des besoins s'élève donc à $16,8.10^6$ m^3/an.

En cycle hydrologique normal, la nappe de DJIBOUTI sera censée fournir $13,7.10^6$ m^3/an (cette valeur sera précisée par les mesures complémentaires suggérées ainsi que par le modèle mathématique) et la nappe de KABA KABA : 3.10^6 m^3/an, les besoins étant pratiquement équilibrés par les ressources.

La mise en exploitation de KABA KABA devrait correspondre aux étapes suivantes correspondant aux besoins évalués de DJIBOUTI.

Année	Besoins DJIBOUTI + Irrigation	Nappe DJIBOUTI	KABA KABA
1988	$14,7.10^6$ m^3/an	$13,7.10^6$	$1,10^6$ m^3/an
1989	$15,2.10^6$ m^3/an	$13,7.10^6$	$1,5.10^6$ m^3/an
1990	$15,8.10^6$ m^3/an	$13,7.10^6$	$2,1.10^6$ m^3/an
1991	$16,3.10^6$ m^3/an	$13,7.10^6$	$2,6.10^6$ m^3/an
1992	$16,8.10^6$ m^3/an	$13,7.10^6$	$3,1.10^6$ m^3/an

En cycle hydrologique sec (probabilité de 10 à 15 %), les ressources de DJIBOUTI seront limitées à 10.10^6 m^3/an. La nappe de KABA KABA ne pourra technologiquement fournir que 3.10^6 m^3/an, soit globalement pour les deux nappes : 13.10^6 m^3/an.

Le déficit des ressources par rapport aux besoins vaut :

$16,8.10^6 - 13.10^6 = 3,8.10^6$ m^3/an, soit 23 % des besoins.

Ce déficit ne pourrait être comblé que par des mesures autoritaires limitant momentanément (saison sèche) la consommation d'eau.

Alternativement on pourrait prévoir une mise en exploitation plus rapide et plus intense de KABA KABA pour pallier aux éventuels cycles secs. Cela nécessiterait un planning de mise en exploitation tel que ci-après :

Année	Besoins DJIBOUTI + Irrigation	Nappe DJIBOUTI	KABA KABA
1988	$14,7.10^6$ m^3/an	10	$4,7.10^6$ m^3/an
1989	$15,2.10^6$ m^3/an	10	$5,2.10^6$ m^3/an
1990	$15,8.10^6$ m^3/an	10	$5,8.10^6$ m^3/an
1991	$16,3.10^6$ m^3/an	10	$6,6.10^6$ m^3/an
1992	$16,8.10^6$ m^3/an	10	$6,8.10^6$ m^3/an

3. 3ème phase : 1992 - 2002

La troisième phase consiste en la poursuite et l'achèvement des travaux d'infrastructure pour l'exploitation de la nappe de KABA KABA qui devrait pouvoir fournir de l'ordre de 10 à 15.10^6 m^3/an (volume précisé par le monitoring de la nappe et les simulations du modèle mathématique).

Les besoins de DJIBOUTI en 2002 sont estimés à $19,3.10^6$ m^3/an, augmentés de 10^6 m^3/an pour les besoins de l'irrigation et industries dans l'aire d'Oued de DJIBOUTI.

En cycle hydrologique normal, la nappe de DJIBOUTI fournira de l'ordre de $13,7.10^6$ m^3/an et celle de KABA KABA le solde, soit 19,3 + 1 - 13,7 = $6,6.10^6$ m^3/an, soit sensiblement moins que le renouvellement annuel moyen.

En cycle hydrologique sec, la production de la nappe de DJIBOUTI sera limitée à 10^6 m^3/an et KABA KABA fournira $10,3.10^6$ m^3/an, ce qui pourrait être supérieur au taux de renouvellement. Comme on peut penser qu'il n'y a pas de problème d'eau salée pour la nappe de KABA KABA (ceci doit être vérifié par les reconnaissances préconisées en première phase), on exploitera au-delà du renouvellement de la nappe dans ses réserves. Le renouvellement de la nappe aura lieu au cours des années pluvieuses pour lesquelles l'exploitation de KABA KABA reste en deça du renouvellement annuel.

En phase critique, pour la nappe de DJIBOUTI, la nappe DE KABA KABA pourrait être utilisée en complément.

PROGRAMME DE MISE EN EXPLOITATION DE LA NAPPE DE KABA KABA

		Cycle hydrologique normal		Cycle hydrologique sec alternatif sans restrictions)	
Année	Besoins DJIBOUTI + Irrigation	Nappe DJIBOUTI	Nappe KABA KABA	Nappe DJIBOUTI	Nappe KABA KABA
1993	$17{,}2.10^6$ m^3/an	13,7	3,5	10,0	7,2
1994	$17{,}6.10^6$ m^3/an	13,7	3,9	10,0	7,6
1995	$18{,}2.10^6$ m^3/an	13,7	4,5	10,0	8,2
1996	$18{,}5.10^6$ m^3/an	13,7	4,8	10,0	8,5
1997	$19{,}9.10^6$ m^3/an	13,7	5,3	10,0	9,0
1998	$19{,}2.10^6$ m^3/an	13,7	5,5	10,0	9,2
1999	$19{,}6.10^6$ m^3/an	13,7	5,9	10,0	9,6
2000	$19{,}8.10^6$ m^3/an	13,7	6,1	10,0	9,8
2001	$20{,}3.10^6$ m^3/an	13,7	6,6	10,0	10,3
2002	$20{,}3.10^6$ m^3/an	13,7	6,6	10,0	10,3

Ceci entraîne que KABA KABA devra être pourvu d'une surcapacité technologique d'exploitation de 56 % pour pallier les cycles hydrologiques secs sans restrictions.
Il va de soi que les chiffres énumérés ci-dessus se doivent d'être précisés par les mesures faites sur la nappe et les simulations du modèle mathématique, ce qui permettra d'optimiser les mesures à prendre du point de vue économique.

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OPERATIONAL TECHNOLOGY FOR HYDROLOGY AND WATER RESOURCES DEVELOPMENT AND MANAGEMENT

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ABSTRACT

The WMO Hydrological Operational Multipurpose Subprogramme (HOMS) provides a systematic institutional framework for the transfer of proven operational technology in hydrology and in water-resource development and management. The technology available in HOMS is presented and transferred in the form of components. Each component is complete on its own for some particular use.

This paper describes HOMS in general and demonstrates in particular how the components and sequences available in HOMS can be used to meet the need for hydrological information such as the requirements of water-resource assessment and hydrological forecasting.

INTRODUCTION

In the last two or three decades there has been great progress in hydrological science, which has in turn influenced the practices of national operational hydrological services. One has only to think of conceptual and other models used for forecasting or simulating stream flow, or computerized data bases giving ready access to large archives of hydrological data to realize the progress achieved in a number of countries since, say, the end of the Second World War.

Much of this progress has been made possible by the development of the digital computer, for analyzing large quantities of data more thoroughly and for organizing data storage more efficiently.

At the same time there has been a need for improved hydrological analyses to allow the more effective development of water resources to meet increasing demands. For many countries, and in particular for developing countries, water resources development for agriculture, for industry, and for domestic supply is a necessary step in the economic development of the country. These countries need access to modern techniques of data observation, collection, processing, storage, analysis and modelling to enable them to improve their water resources utilization.

The many techniques available are described, in a general fashion, in text books, scientific papers, and guides to practice, and supposedly any competent hydrologist, given time, could write the necessary computer programs, or detailed operational manuals needed to put these techniques into practice in his own service. However, time is not generally available and even if it were, the work involved would only be repeating, probably in a less satisfacatory manner, work already carried out by the originator of the technique. Thus, what is needed is a source of proven, operational technology, in the form of computer programs, manuals, and instrument specifications, for carrying out the operations of a hydrological service. To meet this need the World Meteorological Organization established the Hydrological Operational Multipurpose Subprogramme (HOMS).

HOMS is a major subprogramme of WMO's Operational Hydrology Programme and aims to provide an efficient means of technology transfer between national hydrological services. It does this by making proven, operational, hydrological technology available to users in the form of components. These components are of various types: principally computer programs, technical and general guidance manuals, and instrument descriptions and specifications.

The objectives of HOMS are:

- to improve the quality and quantity of hydrological data available for use in water resources and other projects;

- to assist directly such field projects of WMO Members;

- to aid in the application of appropriate hydrological technology and in related training, especially in developing countries;

- to provide an international systematic framework for the integration of the many techniques and procedures used in the collection and processing of hydrological data for water resources systems.

EARLY DEVELOPMENT OF HOMS

HOMS was established by the Eighth World Meteorological Congress in 1979. First proposed by the fifth session of the WMO Commission for Hydrology (CHy) in 1976, it was at the Commission's sixth session in 1980 that the first or pilot phase of HOMS was planned in detail. During the pilot phase Members of WMO were invited to participate in HOMS by establishing a HOMS National Reference Centre (HNRC) and by submitting components for inclusion in HOMS. The steering committee for HOMS, established by CHy-VI as the Advisory Working Group of the Commission, considered the components offered and approved over 300 for publication in the first edition of the HOMS Reference Manual. This first edition, which appeared in August 1981, has since been extended by a series of supplements containing descriptions of new or revised components. Six supplements had been issued by May 1984, and in addition, translations into Chinese, French and Russian had been published. A Spanish translation is in preparation. Following the publication of the Reference Manual, transfer of components could commence. In the first year alone, approximately 200 transfers between national reference centres were notified to the HOMS Office in the WMO Secretariat. This initial response by the national Hydrological Services of WMO Members exceeded the expectations of even the most optimistic proponents of HOMS.

A report on the successful implementation of the first phase was presented to the Ninth World Meteorological Congress in 1983, which approved outline plans for the second phase to run from 1984-1991. These outline plans are to be developed in more detail by the seventh session of the WMO Commission for Hydrology scheduled to be held in Geneva in August/September 1984.

ADMINISTRATIVE ARRANGEMENTS

HOMS National Reference Centres

A country which wishes to participate in HOMS does so by designating a HOMS National Reference Centre (HNRC). These Centres are usually established in the national Hydrological Service, or in a similar body dealing with hydrological matters at the national level. A list of the WMO Members who had established a HNRC at May 1984 is given in Table I. This table also lists regional focal points which have been set up in certain international river basin authorities.

In the resolution establishing HOMS, the World Meteorological Congress suggested that the HNRC should undertake the following activities:

- Establishment of an inventory of components which are currently available and operationally used in the country and which are considered appropriate to be proposed for inclusion in the HOMS project;

- Collection of these components, their adaptation as necessary, and transmission of their description to WMO for inclusion in the HOMS Reference Manual;

- At the request of other countries or of WMO, transmission of these components, either bilaterally or through WMO, for use and application in other countries;

- Receipt and storage of components requested and received from other countries, either directly or through WMO;

- Calling the attention of potential users in the country to the availability of HOMS components;

- Assistance in the use and application of the HOMS components, as appropriate.

The organization and duties of an HNRC is a matter for each country to decide and there is a considerable variety here as these examples show:

Belgium

The HNRC for Belgium, which is the Hydrology Section of the Royal Meteorological Institute, has contributed many components to HOMS, mostly in the fields of primary and secondary data processing. The HNRC has made a special effort to provide excellent documentation both in English and French for its software and this effort has been well rewarded with many requests for its components.

Canada

The HNRC for Canada is the Inland Waters Directorate, a federal agency. It has submitted a large number of components for HOMS, many of which have been supplied, through the HNRC, by other Canadian agencies. The HNRC has taken a co-ordinating role for the development and transfer of hydrological technology within the country. To assist this coordination, the HNRC has set up a publicly available data base with details of all components, thus giving Canadian hydrologists easy access to the information.

Hungary

The HNRC for Hungary is the Research Centre for Water Resources Development (VITUKI). A large number of HOMS components have been proposed by VITUKI for HOMS. One of the problems arising was with the language; as descriptions and users manuals were originally in Hungarian. However, many have since been translated by VITUKI, making them more generally available.

Malaysia

The HNRC for Malaysia is the Drainage and Irrigation Department of the Ministry of Agriculture. It has submitted a number of components giving drawings, etc., for the local manufacture of instruments and has requested components from several other centres, particularly in the field of data processing.

Nordic HOMS Reference Centre

Some countries have decided to join in establishing one HOMS Reference Centre carrying out the functions of HNRC for them. This is the case of Denmark, Finland, Norway and Sweden which have established the Nordic HOMS Reference Centre. This arrangement appears to be quite cost-effective.

China

China has established a HNRC in the Ministry of Water Conservancy and Electric Power with assistance from the UNDP. Similar requests from other countries are under consideration. China has supplied components on hydrometric equipment and design flood estimation and received in return components on data transmission, storage, processing and modelling.

United States of America

The HNRC for the United States of America is the Hydrology Office in the National Oceanic and Atmospheric Administration (NOAA). All major US water-resource agencies are associated with this centre and have supplied components through it. As in some other industrial countries, a very large number of potentially useful components is available, and a problem of the HNRC is to make a suitable choice.

International Co-ordination

The co-ordinating functions carried out at the international level by WMO in the development and implementation of HOMS consist, in particular, in:

- Ascertaining the needs of Members in the overall orientation of the project;

- Developing and distributing to Members general and specific guidance on the substance and form of HOMS components to be prepared at the national level;

- Preparing, maintaining, and updating the HOMS Reference Manual and periodically distributing it to Members;

- Assisting in the technology transfer among Members, within the framework of the project.

This international coordination is carried out by the Commission for Hydrology and its constituent working groups and rapporteurs, supported by the HOMS Office in the WMO Secretariat. The steering committee for HOMS is the Advisory Working Group of the Commission and it is responsible for the development of HOMS, within the framework laid down by Congress and the Commission. A particular responsibility of the steering committee is that of approving all new components proposed for inclusion in HOMS and in this task it is helped by the other working groups and rapporteurs of the Commission who are able to comment, from an expert standpoint, on the technical content of proposed components. To ensure a close co-ordination the responsibility for advising on the development of HOMS in particular technical fields has been written into the terms of reference of the various CHy working groups and rapporteurs.

WMO Secretariat support to HOMS is arranged through the HOMS Office which has a staff of two professional level officers. The Office handles day to day administration of the subprogramme, collects material for, and publishes new supplements to the HRM, assists with transfers of components, and monitors component transfer activity.

TECHNICAL ORGANIZATION

Components, Sequences and User Requirements

As mentioned above, the basic unit of the technology transferred within HOMS is the component.

Components take different forms. Typical examples include a computer program and documentation for a hydrological model, a streamflow gauging manual, or construction drawings for a hydrometric cableway. Each component is self-contained and contains the instructions, or computer programs for carrying some hydrological "operation". In the above examples, the operations would be, respectively, fitting and using the model, measuring the discharge of a river, and installing the cableway. A simple classification system is used to index the components. The subject matter of HOMS is divided into different sections each devoted by a letter (see Table II for a list of sections) and the sections are further divided into subsections each denoted by two decimal digits, again on the basis of subject matter. For example section J deals with hydrological models while subsection J12 is for hydrological models for forecasting. Within each subsection components are available at three levels of complexity; the code 1 is used for the simplest components, 2 for medium, and 3 for the highest level of complexity. A two digit number is then assigned to distinguish components within the same subsection and level of complexity. Thus the full classification number of a component is of the form: H09.2.01. This component, of medium complexity (.2.), deals with the primary processing (section H) of sediment data (subsection H09). It is the first component with this classification.

The classification system is one method of locating the component required for some task. More commonly users will want to use several components to build a system and will need assurance that the components he selected are compatible with each other. For this purpose, sequences of components have been developed. Each Sequence lists a group of components, which can be used to meet some user requirement. In addition User Requirements are available, these list the sequences and/or components needed to solve some major hydrological problem such as flood forecasting, or design flood determination.

This system enables a component to be located by subject matter, (i.e., section and subsection), complexity, and by the purpose it serves, that is by membership in a sequence. This threefold classification may be illustrated by the cube in Fig. 1.

HOMS Reference Manual

This is the basic document of the subprogramme; it describes how HOMS operates and contains a series of annexes listing HOMS National Reference Centres, and describing the components, sequences, and user requirements available. Each component has a two page summary description in a standard format (see the example annexed to this paper). This summary description

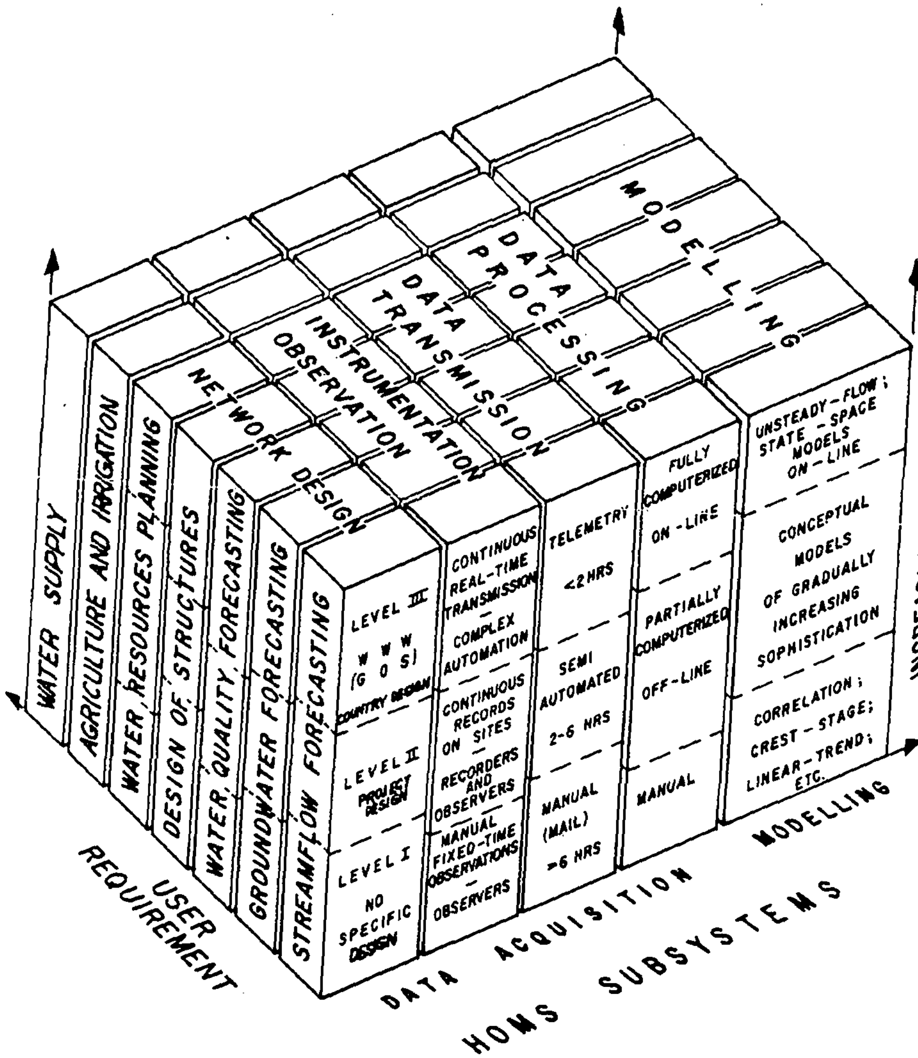

Figure 1. Structure of HOMS

contains information on the component - its purpose, methods employed, requirements and restrictions, etc., which will enable an inquirer to determine whether it will meet his requirements. These summary descriptions, in Annex C to the Reference Manual, are indexed by a list of all available components, ordered by component number, contained in Annex B. Annex B also contains a complete outline of the classification system into sections and subsections.

Sequences are described in Annex F, with an alphabetical index in Annex E. Each sequence description lists the components comprising the sequence with a few lines of description of each. Comments on the purpose of the sequence (i.e., the user requirements it meets) are also given.

Annex D gives the HOMS User Requirements. Each of these show how HOMS might be used to satisfy some requirement for hydrological information.

The list of HNRCs, with their addresses, is contained in Annex A.

OPERATION OF HOMS

This section gives a brief summary of the experience gained in operating HOMS from its establishment up to early 1984.

At that period over seventy Member countries of WMO had signified their intention of participating in HOMS by nominating a HOMS Reference Centre; these countries are listed in Table I.

Component transfers are usually arranged directly between the two HNRCs involved: the requester, and the donor. However, in about one third of cases the request for a component, or components, has been made through the WMO Secretariat. By emphasizing the bi-lateral nature of transfers, and avoiding a large centralized bureaucracy, HOMS ensures a close co-operation between the donor and the requester/recipient.

Table II shows an analysis of the requests for the transfer of components notified to the HOMS Office in the WMO Secretariat by February 1984. Over 500 requests were notified in this period, and approximately two thirds of the components then available were requested at least once. Components dealing with data processing and storage (sections G, H and I) proved popular, receiving 40% of requests. A further 30% of requests were for components dealing with hydrological modelling (section J) and analysis (section K).

CONCLUSIONS

In its first few years of operation HOMS has shown that it meets a real need of the Hydrological Services of WMO Member countries. The initial response was more enthusiastic than had been expected and rate of transfer of components continues to be at a high level.

It is an essential feature of HOMS that the technology available is provided in small units: the components. This enables a user to select the technology he needs, at the level of complexity appropriate to his circumstances. The existence of sequences gives advice on the building of larger systems, if the user should wish to do so. As each component is offered by the originator from the technology he uses himself in his daily operations, there is an implicit guarantee that the technology available through HOMS works and is useful. The success of HOMS is undoubtedly due to its adoption of these principles.

Table I - Members of WMO with a HOMS National Reference Centre.

Algeria	Italy
Argentina	Japan
Australia	Kenya
Austria	Malaysia
Bangladesh	Mauritius
Belgium	Mexico
Bolivia	Nepal
Brazil	Netherlands
British Caribbean Territories	New Zealand
Bulgaria	Niger
Burma	Nigeria
Cameroon	Norway
Canada	Pakistan
Chile	Panama
China	Peru
Colombia	Philippines
Costa Rica	Poland
Cyprus	Portugal
Czechoslovakia	Republic of Korea
Democratic Peoples' Republic of Korea	Romania
Denmark	Saudi Arabia
Ecuador	Spain
El Salvador	Sudan
Ethiopia	Suriname
Fiji	Sweden
Finland	Switzerland
France	Thailand
German Democratic Republic	Trinidad & Tobago
Germany, Federal Republic of	Tunisia
Greece	Uruguay
Guatemala	USSR
Guyana	United Kingdom
Honduras	United States
Hong Kong	Upper Volta
Hungary	Venezuela
India	Vietnam
Iraq	Yugoslavia
Israel	

HOMS Regional focal points.

Centre Interafricain d'Etudes Hydrauliques (CIEH), Ouagadougou, Upper Volta
Comité Regional de Recursos Hidráulicos (CRRH), Tegucigalpa, Honduras
Hydrometeorological Survey of Lakes Victoria, Kyoga, and Mobutu Sese Soko, Entebbe, Uganda
Secretariat of the Interim Committee for Co-ordinating Investigations of the Lower Mekong Basin, Bangkok, Thailand

Table II

HOMS Sections with numbers of components available and requests

Section	Title	No. of components	No. of requests
A	Policy, planning, and organization	1	1
B	Network design	2	4
C	Instruments and equipment	45	38
D	Remote sensing	7	2
E	Methods of observations	34	50
F	Data transmission	17	28
G	Data storage, retrieval and dissemination	16	53
H	Primary data processing	51	74
I	Secondary data processing	38	86
J	Hydrological models for forecasting and design	77	96
K	Analysis of data for planning, design and operation of water resource systems	32	62
X	Mathematical and statistical computations	11	37
		331	533

Note: No. of components as at May 1984 (after supplement 6), requests as at February 1984.

MULTI-VARIABLE RATING CURVE (QTOBBV)

1. Purpose and objectives

To increase the accuracy of the daily discharge data for river reaches with low slopes. The program can also take into consideration the effect of a tributary.

2. Description

The simple two-variable rating curve, Q=f(H), is often used for determining discharges, though the discharge depends not only on the stage but also on the local water slope. This component, which computes the discharge from stage and slope, is particularly useful in cases of low slope. Since the measurement of local slope is difficult, the average slope of a longer river reach, computed from two neighbouring gauges, is usually used. The distance between the two gauges may vary between 2 and 40 kms, depending on the river and, if the gauges are chosen appropriately, the measured slope, S, will be well correlated with the local slope, and can be used to correct the discharge value determined from the two variable, Q-H, rating curve.

The correction factor is equal to the square root of the ratio of the slope, S, to the permanent slope, Sp, which can be found from corresponding stages of flood wave peaks or of low flows on the selected gauges.

The principles of the method may be summarised as follows:

- Determine Sp=f(H) for the gauging section as described above. A tributary between the two gauges can be catered for by using a different Sp-H curve for each tributary stage.

- Reduce measured discharges to permanent discharges by dividing by the correction factor, sqrt(S/Sp).

- Hence determine the permanent rating curve, Qp=f(H), of the gauging station

- Use measured stage values to determine the permanent discharge from this rating and correct to the true discharge by multiplying by the correction factor.

A computer program has been developed for these computations.

3. Input

Rating curve for steady flow conditions. Stage-surface slope equation for steady flow conditions. Daily stages of the given and nearby stations.

4. Output

Daily, monthly and yearly mean and extreme discharge values.

5. Operational requirements and restrictions

A series of discharge and water surface slope measurements are necessary for the predetermination of the equations. Having these, water stage data series of the main and nearby stations are needed. A medium size computer is required to operate program (up to 64 Kbytes).

6. Form of presentation

The program written in FORTRAN IV is available on punched cards or paper tape, or magnetic tape. The instructions are to be found in the program itself as comments.

7. Operational experience

Used by Water Resources Research Centre (VITUKI) H-1453 Hungary.

8. Originator and technical support

See (7) above.

9. Availability

From HOMS National Reference Centre for Hungary.

10. Conditions on use

No restrictions.

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

Paper number 289 Aspect number 2

THE FORECAST OF DROUGHT DISCHARGES OF THE RIVER MEUSE

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ABSTRACT

After an analysis of the drought characteristic discharges of the River Meuse upstream of Liège and a description of all the problems which could arise because of those deficiencies, we have mentioned that the solution of this problem is to build dams upstream of Liège, which could regulate the low flows by releasing the extra flow to maintain the necessary discharge.

As the water supply could take more than ten days to reach the town, the problem is to know how to forecast drought periods.

An analysis of low flows and precipitation records observed during 22 years showed that a regression equation between the discharge in drought period and the rainfall observations of the three preceeding months may be established with a rather good correlation coefficient.

The relation we have found by using a multiple linear regression equation allows us to forecast drought periods 2 or 3 weeks in advance.

Keywords : Drought discharges, River regulation, Flood control, Discharge forecasting.

INTRODUCTION

The object of this paper is to describe a method estimating the discharge of the River Meuse during drought periods.

The problem is indeed very important because the River has an average daily discharge of about 270 m³/s. But, during flood, the flow can raise to 3000 m³/s and it can also fall below 30 m³/s during drought period (figure 1).

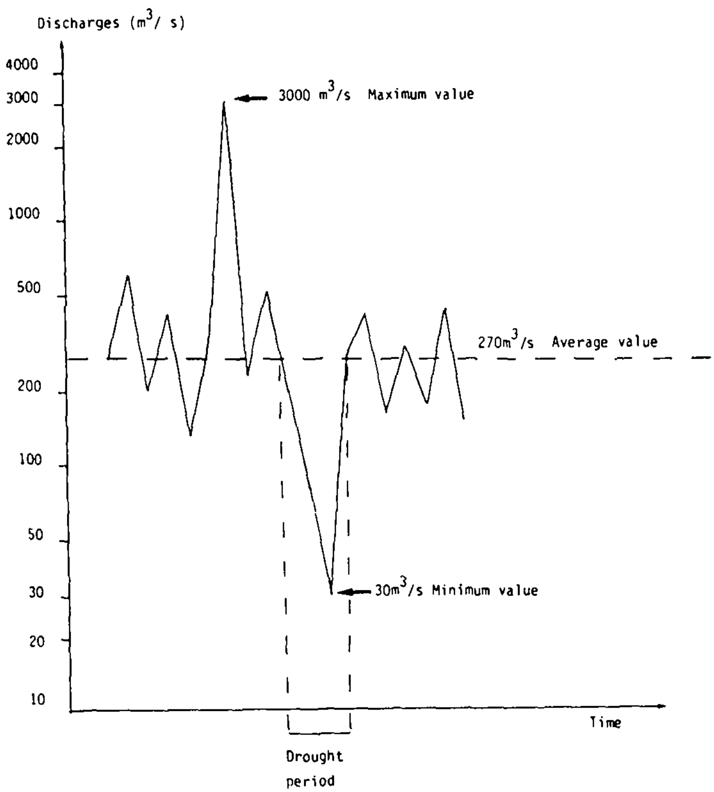

Figure 1 - Discharges variation of the River Meuse at Liège

Table 1 gives the drought characteristic discharges we observed upstream of Liège from 1959 to 1980.

We can see that the daily mean flow in drought periods could be very low and the average value doesn't exceed 38 m³/s.

TABLE 1	GAUGE STATION : AMPSIN-NEUVILLE (Upstream of Liège)
YEAR OF RECORD	DROUGHT DISCHARGES
1959	17 m³/s
1960	44 m³/s
1961	34 m³/s
1962	29 m³/s
1963	32 m³/s
1964	7 m³/s
1965	89 m³/s
1966	72 m³/s
1967	39 m³/s
1968	56 m³/s
1969	38 m³/s
1970	50 m³/s
1971	22 m³/s
1972	39 m³/s
1973	27 m³/s
1974	28 m³/s
1975	33 m³/s
1976	12 m³/s
1977	35 m³/s
1978	36 m³/s
1979	34 m³/s
1980	65 m³/s
Average	= 38 m³/s

PRINCIPAL NEEDS THE RIVER MEUSE MUST SATISFY

As we know, problems could arise because of those deficiencies in the river flow.

Figure 2 gives an idea about the principal needs the River Meuse must satisfy.

First of all, it has to provide a satisfactory supply of water for industry and population (figure 2.1.) then it has to maintain a sufficient water depth for navigation (figure 2.2.), it has also to supply a steady flow in the Albert Canal (figure 2.3.) and finally it has to maintain a minimum discharge in the river when flowing into the Netherlands (figure 2.4.).

Therefore a good knowledge of the minimum flow in this river is essential.

It has been estimated that a minimum of 50 m³/s must be maintained in order to provide all these water supplies [Ref.6].

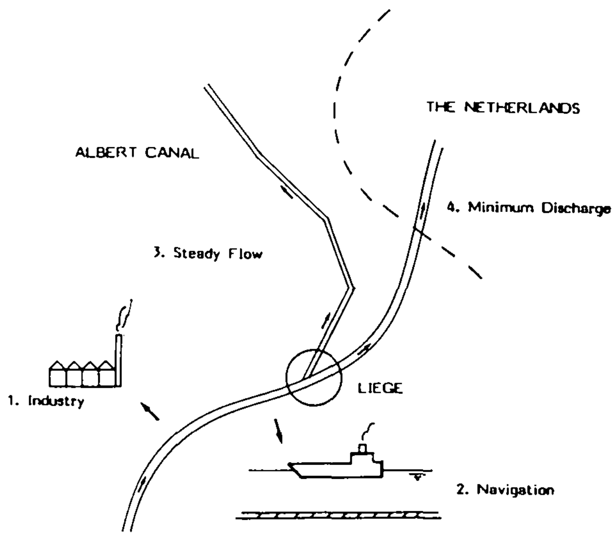

Figure 2 - The principal needs the River Meuse must satisfy

THE SOLUTION THAT COULD BE USED TO REGULATE LOW FLOWS

The solution for this problem would be to build some dams upstream of Liège as shown in figure 3.

These dams could regulate the low flows by releasing the extra flow to maintain the needed discharge.
But this water supply could take more than 10 days to reach the town. So the remaining problem is to know how to forecast drought periods before the discharge at Liège falls below 50 m^3/s.

The drought periods' forecast is not a very difficult problem because as you know, discharges may be considered as a consequence of rainfall.

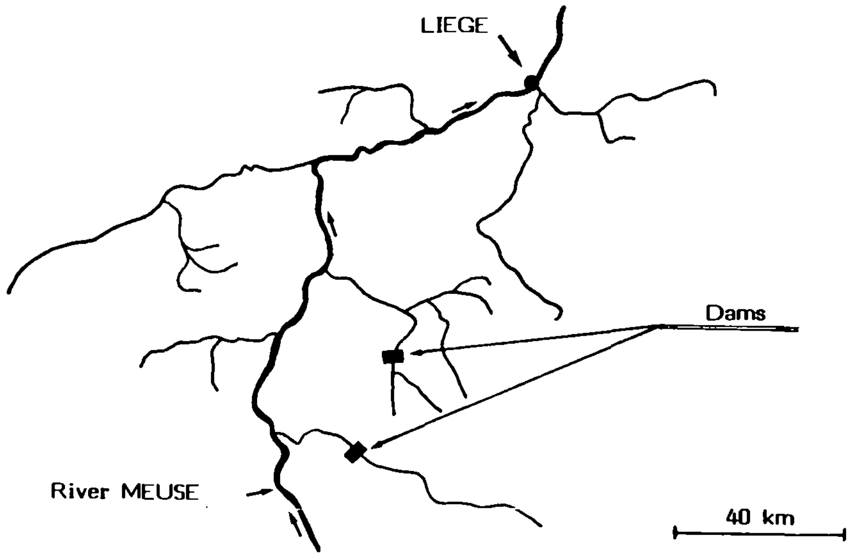

Figure 3 - Regulation Dams to build upstream of Liège

RELATIONSHIP BETWEEN RAINFALL AND DISCHARGES

Let us see if, in this case, there is any relationship between rainfall and drought discharges.

Figure 4 represents the monthly mean discharges observed from 1959 to 1980 upstream of Liège at Ampsin-Neuville. We could see that the drought period goes from the first of May to the end of October, the lowest discharges occuring in August and September.

If we consider now the monthly mean rainfall records, we can see that the precipitations don't vary a lot in the whole year (figure 5). There is only a relative deficit in April, September and October, the maximum occuring in July, November and December.

If we look at the same time at figure 4 and 5, we notice that during low flows quite high rainfall quantities are observed.

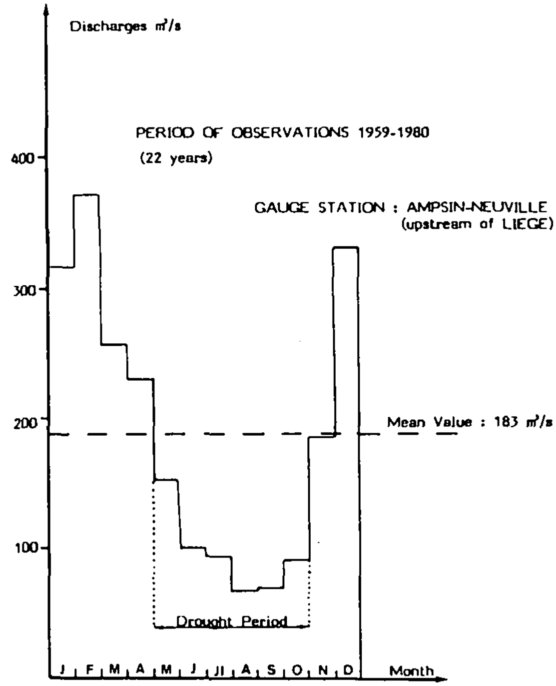

Figure 4 - The monthly mean discharge of the River Meuse

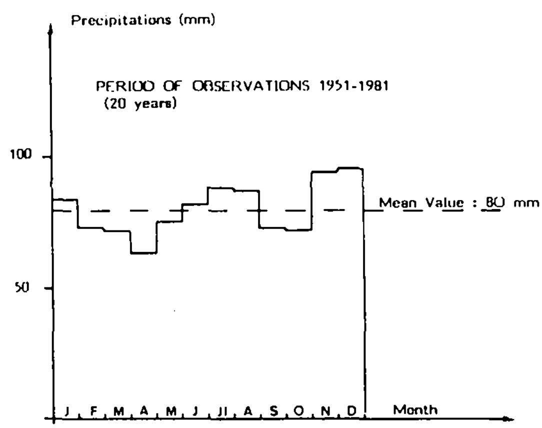

Figure 5 - The monthly mean rainfall records

This situation doesn't seem normal but in fact, it is, we musn't forget that during this period evaporation takes off a large amount of the available rainfall water.
Consequently, before looking for any correlation between precipitations and discharges we have to withdraw the monthly mean values of evaporation from the rainfall diagram.
If we do it, we get a very different pattern which is the real distribution of rainfall water excess all the year long (figure 6).

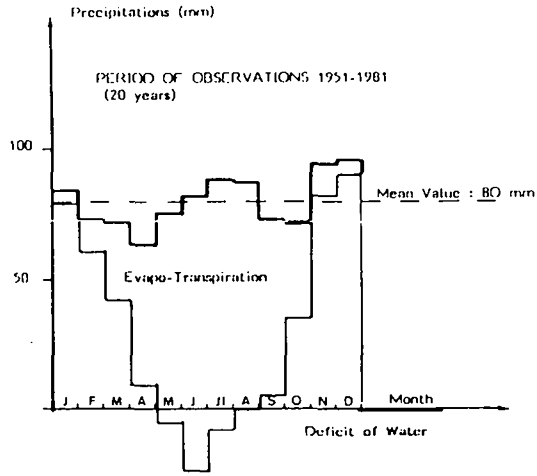

Figure 6 - The monthly mean values of water excess

In this case, we see that there is a real deficit of water in May-June and July. This period corresponds exactly to the drought period we have already noted in figure 4.

On the other hand, we could say that drought discharges occur 3 months after the first deficit of water. This is a very important fact which will allow us to forecast drought discharges using only the rainfall observations of the 3 preceeding months.

THEORETICAL SOLUTION

According to the observations we have mentioned in the last paragraph, let's assume there are no precipitations during the three preceeding months.

The discharge of the river will vary as shown in figure 7 according to this base-flow-curve and the discharge at any time may be computed very nearly by using this equation :

$$Q = Q_o e^{-\alpha t}$$

where Q_0 is the discharge at the begining of the drought period
Q^0 the discharge at time t
and α the coefficient of aquifer

If any precipitation occurs during this period, the continous discharge curve will change as shown in figure 8.

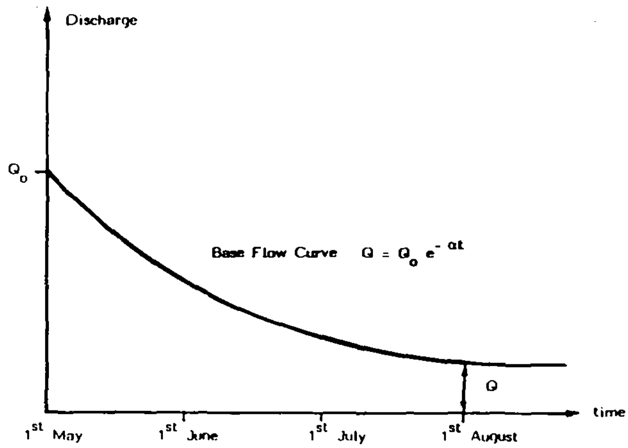

Figure 7 - Base Flow Curve

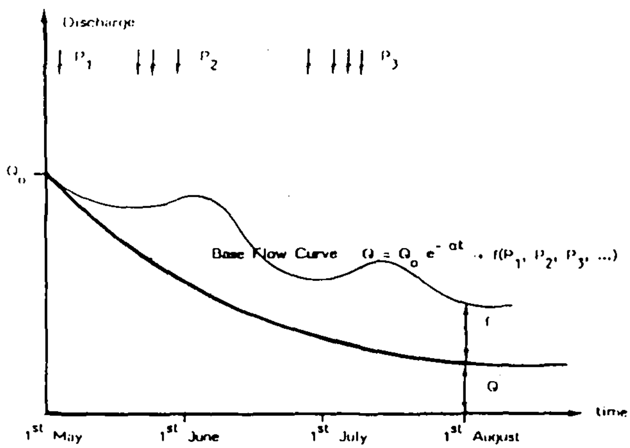

Figure 8 - Discharges-time variation Curve

The discharge at time t (that means in August) could be computed by using this equation

$$Q = Q_o e^{-\alpha t} + f(P_1, P_2, P_3, ...) \qquad (2)$$

where f is a fonction of all the precipitations which have been observed during the preceeding three months. This last function may be obtained by establishing a multiple linear regression equation with 4 variables.
In this case, the general equation can be written like this :

$$Q = Q_o e^{-\alpha t} + A\,P_m + B\,P_j + C\,P_{jl} + D \qquad (3)$$

where Q is the dependent variable (or the drought discharge)
P_m, P_j, P_{jl} are the independant variables (or the monthly rainfall recorded in May, June and July
A, B, C are the multiple regression coefficients.

ANALYTICAL SOLUTION

In equation n° 3 there are four parameters A, B, C and D to be determined. The residual of Q (or the difference between the observed value of Q and the estimated one) is :

$$\underbrace{r\,Q}_{\text{Residual of Q}} = \underbrace{Q}_{\text{Observed value}} - \underbrace{Q_o e^{-\alpha t} + A\,P_m + B\,P_j + C\,P_{jl} + D}_{\text{Estimated value}} \qquad (4)$$

By use of the "least-squares method" to minimize the sum of rQ^2, we obtain the following partial deferential equations :

$$\begin{aligned}
&1. \quad A\,\Sigma(\Delta P_m)^2 + B\,\Sigma(\Delta P_m.\Delta P_j) + C\,\Sigma(\Delta P_m.\Delta P_{jl}) = \Sigma(\Delta Q\,.\Delta P_m)\\
&2. \quad A\,\Sigma(\Delta P_m.\Delta P_j) + B\,\Sigma(\Delta P_j)^2 + C\,\Sigma(\Delta P_j.\Delta P_{jl}) = \Sigma(\Delta Q\,.\Delta P_j)\\
&3. \quad A\,\Sigma(\Delta P_m.\Delta P_{jl}) + B\,\Sigma(\Delta P_j.\Delta P_{jl}) + C\,\Sigma(\Delta P_{jl})^2 = \Sigma(\Delta Q\,.\Delta P_{jl})\\
&4. \quad \bar{Q} = \bar{Q}_o e^{-\alpha t} + A\,\bar{P}_m + B\,\bar{P}_j + C\,\bar{P}_{jl} + D
\end{aligned} \qquad (5)$$

where $\bar{P}$ and $\bar{Q}$ are the main values
P the difference between P and $\bar{P}$
and Q the difference between Q and $\bar{Q}$

The solution of these equations gives the four coefficients A, B, C and D.

By this analysis the drought discharge of the River was found to be :

$$Q_{august} = 0,31\ Q_o + 0,48\ P_m + 0,385\ P_j + 0,619\ P_{jl} - 76,749 \qquad (6)$$

This equation allows us to compute the discharge in August knowing only Q_o (the base flow at the begining of May) and the mean monthly rainfall in May, June and July.

To test the correlation degree between the estimated discharge and the observed one we have computed the correlation coefficient and we have found 0.825. This means that the association degree of the variables is rather good.

CONCLUSION

As a *conclusion*, we can see that we have a rather good relationship between the drought discharges and the rainfall of the three preceeding months. The relationship we have found allows us to forecast the mean flow in august, only knowing the precipitations observed from May to July.

The great advantage of that calculation method is the easiness with which it can be applied.

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	WATER RESOURCES FOR RURAL AREAS AND THEIR COMMUNITIES

Paper number 297 Aspect number 10

WATER POLICIES: REGIONS WITH INTENSE AGRICULTURE

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ABSTRACT

This paper outlines one of the studies being carried out by the "Regional Water Policies Project" at the International Institute for Applied Systems Analysis (IIASA). It considers agricultural regions where both groundwater and surface water systems are interacting elements of the environment. For experimental purposes, it is based on an agricultural region in the Netherlands and is conducted in close collaboration with institutes in the Netherlands and the USSR.

The activities of different water users in the considered region have started to interfere with each other to such an extent that if present trends continue, tnis could could have severe repercussions not only on nature areas but also on the regional economy.

The study is focused on the analysis of regional regulation policies providing for a satisfactory balance between agricultural development and the sustainable evolution of the environment in the long run. Methodologically, it is based on the use of a two-stage decomposition analytical approach, which includes scenario analysis and policy analysis that consider the behavioural aspects of the users more explicitly. As far as implementation is concerned, the study aims at elaborating systems of interlinked mathematical models of economic and environmental processes. Embedded in an interactive computer software, these systems are designed as a supplementary tool to be used by regional decision-makers in their analyses of possible directions of regional development.

INTRODUCTION

Intense socio-economic development in many regions of the world puts an increasing pressure on the environment both by consuming natural resources and by discharging pollutants that are hazardous to the population and to natural ecosystems. A substantial part of these impacts takes place through regional natural water systems. Apart from being a resource that is vital for socio-economic development and for the evolution of natural ecosystems, the regional water system is a basic medium through which local human interventions penetrate to and are "felt" in other parts of the region and also frequently beyond its boundaries.

In different regions different types of economic activities can vary in degrees of their influence on water systems. Here we consider regions where agriculture is the dominating activity both in its economic value and in the degree of its impacts on the regional surface and groundwater systems and through them on the whole regional environment as well as on other activities.

Clearly, other water-users in an agricultural region, like industry, the population at large, etc., should also be considered. But rather than going into their structural details, it suffices to consider them only in terms of demands that they make on the quantity and quality of water resources, assuming these demands as exogenously fixed.

CONCEPTUAL AND METHODOLOGICAL FRAMEWORK

Hierarchical institutional structure of a regional socio-economic subsystem

We view a regional system under study as consisting of two major parts: the environmental subsystem and the socio-economic subsystem. These subsystems interact with each other in a variety of ways, and in the majority of real systems these interactions lead to the deterioration of the environmental subsystem which is potentially dangerous for the existence of the whole socio-economic-environmental system in the long run.

Typically, regional socio-economic subsystems are of a complex hierarchical structure. They include interdependent elements (producers-users of the environmental resources, various legislative agencies, governmental and regional commissions, etc.). Each of these elements has its own preferences and possibilities for action to influence the evolution of the whole system.

The lower-level elements (users of the environment) of this subsystem are those directly interacting with the environment. These interactions depend upon the production technologies (or, generally, the environment use technologies) implemented by the users, and they use these technologies according to their preferences. Depending on the region, the use of these technologies may involve land-use practices, irrigation and other water-use practices, waste-disposal practices, and many others. The major fact is that in regional systems these local interactions are often focused on local goals and are not coordinated with each other.

On the other hand, the upper-level elements of the socio-economic subsystem (governmental, regional agencies, etc.) have preferences more closely reflecting the regional perspectives. These preferences may be related to various aspects of the regional development and reflect the goals of different agencies and regional interest groups.

The upper-level elements of the socio-economic subsystem do not directly control the interactions of lower-level users with the environment, but may have varying degrees of regulation power (depending on the particular region or problem) for influencing their behaviour indirectly using economic, legislative and/or other types of policies or mechanisms. These policies may include imposing constraints on the use of surface water and groundwater, on the amounts of fertilizers used in different parts of the region, various economic measures like pricing, taxing, subsidizing and others. The feasibility of various regulation policies depends on the structure of the particular socio-economic subsystem considered and therefore adequate understanding is needed of this structure and also of the preferences and possibilities of its interacting elements.

No formal description can encompass all the aspects of a real socio-economic subsystem. In this study, we use a simplified two-level representation of the socio-economic subsystem of the form shown in Figure 1.

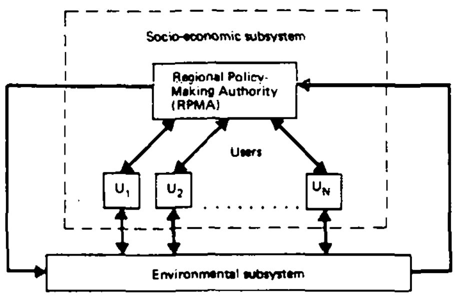

Figure 1. Regional hierarchical structure

We assume that the upper-level element of this structure (regional policy making authority, RPMA) represents the regional perspectives and has at its disposal policies capable of influencing to a certain extent interactions of the lower level elements (producers-users) with the environmental subsystem. It is the presence of this upper-level element with a certain degree of regulation power that distinguishes regional systems of our interest.

Decomposition analytical approach

Apparently, an accurate formalization and analysis of the above two-level structure of the socio-economic subsystem requires simultaneous consideration of preferences and actions (and reactions) of all of its interrelated elements. This formalization based on the concepts of the hierarchical game theory (see, Vatel and Ereshko, 1977; Germeyer, 1976) helps conceptualize and understand the nature of regulation policies and decision processes in systems of this type, and also indicate the lines of the analysis. But being certainly useful conceptually and methodologically this formalization is often too complex for its straightforward computational implementation. Recognizing this, we use in this study a heuristic approach based on what may be referred to as two-stage decomposition of the analysis.

The first stage of the analysis using this approach is directed towards generating trajectories of the potentially rational development of the system under study. No behavioural aspects of the lower-level elements are considered explicitly at this stage, and the analysis results in generating in some sense a reference trajectory of regional development. This trajectory is based on trade-offs among goals of different regional interest groups. We call this trajectory a reference scenario of regional development. This scenario is described in terms of the essential parameters of the socio-economic and/or of the environmental structure of the system. Various approaches to regional planning with the explicit consideration of the environmental processes may be used for this analysis.

After having determined a reference scenario, the second stage of analysis is concerned with the search for those feasible regulation policies that influence the behaviour of the users and by doing that can direct the development of the whole system along the lines specified by the reference scenario obtained at the first stage.

Since the first stage of the analysis is performed without explicitly considering feasible regulation policies, the scenario obtained at the first stage may be practically unattainable, or, in other words, no one of the feasible policies may provide for the realization of this scenario. Moreover, feasible policies may differ from each other in their "degree of feasibility" (for example, two policies may differ from each other by the public reaction to their implementation). Recognizing these factors, an environmentally and/or economically less effective scenario may have to be considered that may be achieved using those "more popular" regulation policies.

Schematically, this decomposition analytical procedure is illustrated in Figure 2.

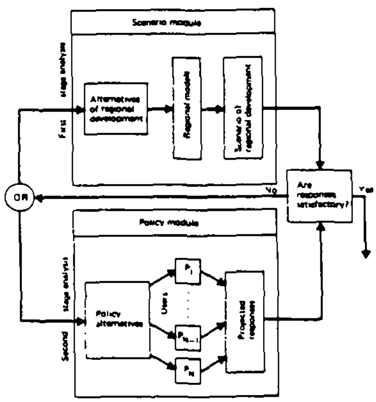

Figure 2. Schematic of decomposition approach.

The scenario module is designed for the analysis and generation of trajectories of future regional development which are potentially rational from the viewpoint of the overall regional perspectives. It should consist of an integrated system of "simple" submodels with which fast screening analyses can be made. Simple submodels should be supported by more comprehensive models for the more accurate estimation of "promising" scenarios.

After having given a short description of the region that this study focuses on as a practical example, we give an outline of the concrete realization of an integrated system of submodels to be used in the scenario module.

THE SOUTHERN PEEL REGION

Environmental setting

The Southern Peel is an undulating area of about 30,000 ha in the south of the Netherlands. The lie of the land varies in altitude between 17 and 35 m above sea level.

A major feature of the hydrogeology is the presence of a fault that divides the area into a Western part - the "Slenk" - which has a deep hydrological basis at 300-500 m below ground level, and an Eastern part - the "Horst" - which has a shallow hydrological basis at 8-36 m.

A large part of the area used to be covered by a layer of peat that grew as a consequence of extremely high groundwater levels. Most of the peat has been delved and used as fuel for heating. The remaining peat areas are now protected from exploitation, because of their value as recreation or nature areas. The nature areas can only keep their value if high enough groundwater levels are maintained. This is because the vegetation has a high water demand that is partly supplied by capillary rise of moisture from the groundwater to the rootzone; a groundwater level lowering would reduce the capillary rise, and the vegetation would suffer from water deficits in dry years. A groundwater level lowering would also increase the soil aeration, causing oxidation of peat, thereby releasing mineralized forms of nitrogen and phosphorus. This eutrophication would improve circumstances for the introduction of species with a high nutrient demand, thus endangering the continued presence of the "natural" species (Kemmers and Jansen, 1983). Eutrophication can also take place by upward seepage of nutrient-rich groundwater.

Human activities and their impacts

Roughly half of the agricultural land is used as pasture for dairy cattle; the remaining area is used for growing a variety of crops, of which maize is the most important one, followed by sugarbeets, potatoes and cereals.

Farmers try to reduce moisture deficits by water conservation, subirrigation, and sprinkler irrigation. The practised water conservation consists of raising water levels in drainage ditches at the end of spring, to reduce the outflow of groundwater; this increases the availability of moisture for capillary rise to the rootzone of crops. The same is achieved by subirrigation, which is the infiltration of imported surface water in the bottoms of ditches, thereby raising groundwater levels in neighbouring fields. Sprinkling is a more direct way of supplying water to the soil. Water for sprinkling is pumped from the groundwater or taken from the surface water supply system. This pumping from groundwater affects agricultural production in other parts of the region and also the conditions in nature areas. In the Southern Peel, the surface water supply system coincides with the drainage system. It consists of some larger canals and a network of ditches and brooks with a varying density.

As is characteristic of all regions with intense agriculture, farmers in the Southern Peel attempt to optimize the nutrient supply conditions of their crops. Both chemical fertilizers and animal slurries are used for this purpose. The urge to heavily fertilize agricultural land stems, however, not only from considerations of optimizing the nutrient supply of crops, but also from the circumstance that in the Southern Peel there is a tradition of factory farming. This factory farming produces large quantities of slurry from cattle, pigs and chickens. These quantities cannot be disposed of by fertilization at the optimal level. This has resulted in heavy over-fertilization of maize fields and dumping of slurry on fallow pieces of land. (Over-fertilization normally causes a decrease in crop production, but maize is not so sensitive to it.) The soils in the Southern Peel are sandy and therefore have poor purification and fixation capabilities. So the excess nitrate is easily leached, thus increasing the nitrogen load on groundwater.

Most of the surface water pollution by agriculture is, however, through surface runoff that has high concentrations of nutrients. Spreading animal slurries on fields in autumn greatly increases the concentrations of nutrients in the surface runoff that is caused by the winter rains.

Water for the population, industry, and factory farming is extracted from the aquifers in the Slenk area by public water supply companies. The resulting lowering of groundwater tables decreases the productivity of agriculture and creates deteriorating conditions in nature areas.

The quality of the extracted groundwater is still excellent, and nitrate levels in wells are hardly increasing yet. (The European Economic Community has stipulated that 50 mg/l is the highest nitrate level that is acceptable. Also, it has prohibited the mixing of low-concentration with high concentration water. This makes it easier to check on the compliance with the stipulated maximum level.) But measurements in phreatic aquifers under agricultural lanas indicate that the concentrations in water "that is on its way to the wells" are alarming, and that action to curb the nitrate pollution of groundwater is urgently needed (Drent, 1983).

A schematic diagram of the main impacts of human activities in the Southern Peel is given in Figure 3.

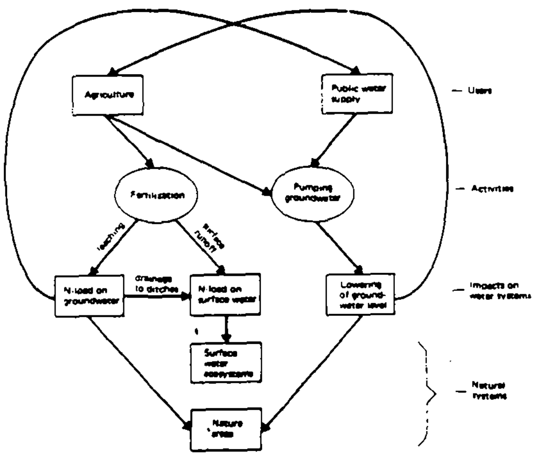

Figure 3. Main impacts of human activities.

SCENARIO MODULE (FIRST-LEVEL MODELS)

Introduction

The scenario module takes into account the following interrelated processes:

1. Agricultural production, economic development of agriculture in terms of incomes, consumptions, savings, investments, changes in land use, changes in factory farming, changes in farm management practices.
2. Water quantity processes: flow in the unsaturated zone, i.e. in the soil, groundwater flows, overland flows (surface runoff), and surface water flows.
3. Soil nitrogen processes: fertilization of soils by application of chemical fertilizers and animal slurries, nitrogen mineralization of slurries, leaching of nitrate to groundwater, and denitrification processes.
4. Water quality processes: transport of solutes in groundwater and surface water.

Common to all the proposed first-level models is that they use a time-step of half a year. The spatial resolution for the models is provided by a division into 31 subregions based mainly on classes of groundwater conditions.

Technologies

We use the term technology for a combination of agricultural activities involved in growing and processing of a certain crop and/or livestock. We assume that technologies differ from each other by their outputs and also by the inputs required to produce these outputs. For convenience, we distinguish between agricultural technologies that use land and those that do not. (See also Vreke and Locht, forthcoming.)

All technologies considered are explicitly characterized by the following types of inputs (resources): labour, capital, water. Land-use technologies are additionally characterized by the input of nitrogen supplied by fertilization. (The reason for explicitly considering only nitrogen in our model lies in that in the region considered nitrogen compounds contained in excessive quantities of animal slurries produced, are also the major pollutants of groundwater.)

Each technology is also characterized by the output or production of the respective goods (crop yields, livestock products). Technologies that involve livestock are additionally characterized by outputs of animal slurries produced as byproducts.

The use of agricultural technologies is described in terms of their intensities. For land-use technologies intensities have the meaning of areas of land allocated to these technologies. For technologies that do not use land and that involve livestock intensities have the meaning of a number of livestock-heads; for a technology not involving livestock the intensity may have the meaning of, for instance, the amount of pig slurry transported to outside the region.

We assume that such inputs as labour and capital for every technology can be represented by corresponding quantities per unit of its intensity. We can also quite adequately assume that the water inputs for technologies not using land can be quanitified in the same normative way (amount per unit intensity).

But the situation is different with describing water inputs and the corresponding outputs for land-use technologies. One reason for this difference is that both the water availability and the output of land-use technologies depend on weather conditions. Another reason is that the availability of water is also influenced by activities in the region, especially pumping of groundwater. In order to take into account the respective possible variations in the performance of land-use technologies, we consider a finite number of options foreach such technology, which cover a suitable variety of typical water availability situations in each subregion. For the sake of brevity we use the term subtechnology to refer to such an option.

Each of subtechnologies k is characterized by the crop productivity cp_k, by the corresponding seasonal averages of the soil moisture vr_k and of actual evapotranspiration ea_k, as well as by the total nitrogen requirement nr_k (all amounts per unit area of land). The value vr_k is treated in our model as the "demand" for soil moisture, the satisfaction of which (together with the satisfaction of the requirement for nitrogen) guarantees obtaining the crop productivity not lower than cp_k. We should note here that the three interrelated parameters cp_k, vr_k, and ea_k are weather dependent, and therefore should be treated as uncertain parameters in the analyses.

Water quantity processes

In the present conception the dynamics of groundwater processes is described by linear equations that have a (stochastic) basic component related to the unperturbed state of the system and components for the influence of control variables (e.g. the groundwater extractions for public water supply). The basic component not only takes into account the influence of meteorological conditions but also physical circumstances that are not influenced by the control variables. For instance, the groundwater level at the end of summer, one of the key variables in the model, is assumed to depend on the extractions for public water supply during summer, on the extractions for sprinkling from groundwater, and on the amounts of subirrigation.

As a selection principle for "discarding" sets of subtechnologies that are not feasible owing to the limited availability of moisture in the rootzone, the requirement is made that the average total moisture content of the rootzone in a subregion must not be lower than the total moisture contents required by the separate subtechnologies. The average total moisture content of the rootzone in a subregion is described by a water balance equation which includes a groundwater level dependence function for the capillary rise.

Nitrogen processes in the soil

Fertilization, mineralization of organic-N

Each technology that uses land has a specified level of the amount of nitrogen that is required for crop growth. This nitrogen can come from different sources, i.e. chemical fertilizer and various types of animal slurries. The nitrogen in slurry and in the soil is present in different forms. Some of it is already mineralized, some of it is bound in easily degradable organic compounds that are rather stable. The first fraction is immediately available after application of the slurry, the second fraction in the course of the first year after application, and the third fraction only in subsequent years.

In the model, we do not include the dynamics of the third fraction. Instead, we assume that the soil content of stable nitrogen is in a "steady state" corresponding to the slurry application in a certain year. In this steady state, the amount of stable nitrogen remains constant; so the amount of stable nitrogen that is mineralized must equal the amount that (yearly) is added by application of slurry. As described by Lammers (1983), it is then possible to compute the amount of nitrogen available for crop growth by simply multiplying the slurry applications by nitrogen effectivity coefficients.

Water quality processes

Groundwater quality

The "simple" model for groundwater quality processes is based on the following assumptions:

1. All deep aquifers over the whole (Slenk) region can be regarded as one mixing cell. The phreatic aquifers are separate mixing cells that overlie the deep aquifers.
2. Decomposition of nitrate in the deep aquifers can be taken into account by a factor α (that depends on the organic matter content of the subsoil).
3. Adsorption and dispersion can be neglected.
4. The volume of water in the aquifers does not change.

Surface water quality

In summer, surface water quality is mainly influenced by the quality of water that is imported from outside the region and by the discharge of waste waters from households, dairies and sewage plants. Owing to the long residence time of surface water in the summer, it is extremely difficult to model water quality processes in that period. This, and in view of the fact that fertilization practices are not of much influence on surface water quality during the summer has led us to for the moment exclude attempts at modelling the latter. Regression formulas are used for describing the concentration of nitrogen in surface runoff during winter as a function of chemical fertilizer and animal slurry applications (Steenvoorden, 1983). The concentration of nitrogen in phreatic groundwater that drains to surface water during winter follows from the groundwater quality model.

Public water supply

If the demands of public water supply are given, then the total of the extractions in the subregions must satisfy respectively for the winter and summer period these demands plus the amount required for factory farming.

Natural ecosystems

Ecological processes in nature areas are influenced in a complex way by the groundwater regime. We do not attempt to describe the dynamics of these processes. Instead, we assume that the conditions in "critical" years, e.g. years with "dryness" that on average occurs only once per decade, are good indicators for the level of satisfaction of water demand of nature areas. Details of ongoing research regarding the influence of the groundwater regime on natural vegetations are given in Kemmers (1983) and Jansen (1983).

The concentration of nitrogen compounds in surface water has a great influence on surface water ecosystems. So if a certain "value" of surface water ecosystems is demanded, the nitrate concentration must meet certain water quality standards.

CONCLUDING REMARKS

This paper outlines the methodological and conceptual framework for the analysis of regional policies providing for a balanced socio-economic development and evolution of natural ecosystems.

Analytically, the paper focuses more on the first stage analysis that is concerned with the generation of potentially satisfactory scenarios of regional development. An important part of this stage is based on the use of relatively simple mathematical models designed for the first level screening analyses.

The screening analyses of scenarios in this study using these models are to be based on the iterative use of various techniques like multiobjective programming using algorithms coupled with simulation runs and also with statistical and other, less formal, procedures for the evaluation of the results at various steps of the analysis. And rather than involving all the above models in each of these analytical steps, we will more flexibly use only some of them with optimization or more generally multiobjective programming algorithms, leaving the others for subsequent simulation runs. The major concern here is that the structure of the analytical procedures involved should enable the analyst to obtain meaningful results in a relatively short time. And as is common to all systems analytical studies the final structure of such workable procedures will emerge in the course of the experimental work with the models briefly outlined in the paper.

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RECONSTRUCTION DES DEBITS MENSUELS D'ETIAGE DE PETITS BASSINS VERSANTS

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RESUME

L'étude commence par un bref rappel des différentes approches des conditions de vidange d'un bassin versant et se poursuit par la présentation d'un modèle utilisé pour la reconstitution des débits d'étiage sur des bassins de surface inférieure à 100 km^2.

Le modèle employé est un modèle déterministe par sa fonction de production et analytique par l'identification de la réponse impulsionnelle (RI) du bassin versant.

L'entrée du modèle est constituée par les pluies "efficaces" au sens de la recharge des nappes profondes. La sortie du modèle est constituée par la série des débits minimums mensuels. La réponse impulsionnelle fonction de transfert du modèle, est identifiée par déconvolution.

Le calage effectué sur cinq bassins versants conduit à une réponse impulsionnelle plurimodale établie sur une durée de 12 à 17 mois. Le premier pic de la RI est invariant dans sa forme, mais sa valeur est directement liée à la surface du bassin.

Une discussion sur une éventuelle signification géologique des autres pics est proposée.

La définition d'une RI synthétique doit permettre d'utiliser le modèle pour la prédétermination des débits d'étiage dans le cas de bassins versants non jaugés.

Mots clefs: conditions de vidange, réponse impulsionnelle, débits minimums mensuels, réponse impulsionnelle synthétique, prédétermination des débits d'étiage.

INTRODUCTION

La détermination des débits d'étiage passe par l'appréciation qualitative des eaux dites souterraines. Ces eaux ont un temps de passage moyen de plusieurs mois au sein du ou des aquifères concernés. Une meilleure définition du "transfert à long terme" d'un "volume impulsion" constitue une approche dynamique de la loi de vidange d'un aquifère.

Ce type d'approche est proposé par D. Poitrinal et G. de Marsily (1973). Cette méthode présente un avantage non négligeable sur la méthode des courbes de tarissement (Pereira, 1977) dont les caractéristiques se trouvent être fortement influencées par l'importance de la charge hivernale.

1. BUTS ET OBJECTIFS

Ce travail s'inscrit dans le cadre global d'une étude d'estimation des débits d'étiage sur des cours d'eau sans mesure directe. Le modèle présenté dans cette étude a été dégagé au terme de la synthèse méthodologique. Il sera appliqué par la suite sur un grand nombre de petits bassins versants pour permettre de forger un outil utilisable directement par le praticien pour l'estimation des débits d'étiage.

Le but de cette étude particulière est de décrire le modèle et de présenter ses premiers résultats de test.

Le modèle développé est un modèle déterministe par sa fonction de production et analytique par l'identification de la Réponse Impulsionnelle. Ce modèle a été sciemment simplifié au pas de temps mensuel pour limiter le volume de données nécessaires à son utilisation ultérieure.

Une paramétrisation ultérieure de la RI, ainsi que l'étude des relations de ses paramètres constructifs avec les paramètres morpho-géologiques du bassin versant permettront la spatialisation de la variable temporelle d'étiage employée.

2. ORIENTATIONS PRISES PAR LES CHERCHEURS

Un chercheur hongrois, E. Mosonyi (1948) a publié une étude portant sur les conditions annuelles de la décharge des aquifères. Il définit un coefficient fonction de la capacité maximale d'emmagasinement du bassin versant et de la lame d'eau écoulée annuelle. Celle-ci est représentative de l'historique de la pluviométrie pour la période de tarissement envisagée.

Dans une application effectuée sur 12 bassins versants des Carpathes, il a trouvé une relation de type logarithmique entre ce coefficient et la surface du bassin versant. Mais les courbes de variation ainsi trouvées étaient spécifiques au type de roches rencontrées et surtout à leur niveau de perméabilité.

Une analyse des conditions de vidange des aquifères est possible par l'analyse des courbes de tarissement (L.S. Pereira, 1977). La méthodologie proposée dégage les caractéristiques statistiques principales de l'ensemble des segments de tarissement. La courbe moyenne ainsi définie s'appelle "courbe caractéristique de tarissement". Il existe cependant, pour un bassin donné, plusieurs courbes caractéristiques typiques d'époques climatiques.

On peut distinguer plusieurs époques climatiques qui se définissent soit sur une base calendaire comme l'avait fait(Pereira 1979),soit d'après les conditions climatologiques (S. Ballerini, 1982).

Depuis une bonne dizaine d'années, deMarsily et Poitrinal (1973) proposent une méthode globale d'analyse basée sur l'utilisation de valeurs climatologiques et hydrologiques en correspondance. Le résultat principal en est la réponse impulsionnelle (ou fonction de transfert) du bassin versant.

Utilisée au pas de temps mensuel par exemple, cette réponse schématise l'hydrogramme de passage à l'exutoire du volume d'eau souterraine résultant d'une impulsion unitaire de pluie de recharge. Une telle méthode permet une approche dynamique de la loi de vidange à long terme d'un aquifère.

3. PRINCIPES DU MODELE UTILISE

La méthode globale proposée se décompose en trois phases distinctes successives.

1ère phase: Définition de l'entrée du modèle.
Dans cette phase, on définit la série des valeurs climatologiques (pluies efficaces) constituant les impulsions de recharge de l'aquifère.

2ème phase: Définition de la sortie du modèle.
Dans cette phase, on recherche la série des valeurs hydrologiques (débits de base) constituant la réponse du système aux impulsions.

3ème phase: Identification de la réponse impulsionnelle.
Mise en relation des séries concomitantes et recherche d'une réponse impulsionnelle spécifique du bassin versant à l'aide d'une technique analytique.

3.1 Définition de l'entrée du modèle

Le calcul de la pluie mensuelle "efficace au niveau de la recharge des nappes profondes" s'effectue par un modèle déterministe simple à réservoir unique (D. Thiery, 1977). La pluie efficace calculée représente une estimation de la fraction de la pluie brute alimentant directement la nappe de l'aquifère du bassin. Le principe schématisé du bilan hydrique mensuel servant de base au calcul de l'entrée (PEFF) est représenté à la fig. 1.

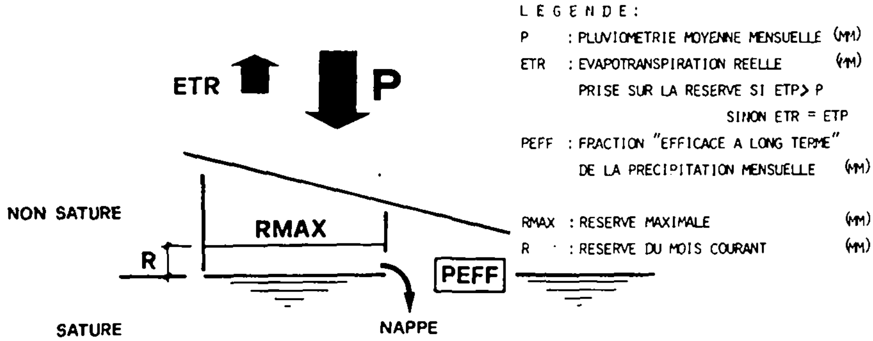

Fig. 1 :Principe schématisé du bilan hydrique mensuel générateur de la valeur d'entrée "PEFF" du modèle

3.2 Définition de la sortie du modèle

La valeur du débit minimum mensuel a été choisie comme variable caractéristique des étiages à cause de sa simplicité d'acquisition. Ce choix présente l'inconvénient d'inclure des eaux de ruissellement dans les périodes de charges hivernales des aquifères, mais l'incidence de cet inconvénient est faible si l'on s'intéresse aux débits de basses eaux estivales (par exemple étiages du Plateau suisse).

3.3 Identification de la réponse impulsionnelle

Parmi les différentes méthodes analytiques d'identification de cette réponse impulsionnelle, la méthode choisie pour notre étude est celle du Centre d'informatique géologique de l'Ecole des mines de Paris qui consiste à développer une réponse impulsionnelle sur une base de vecteurs orthogonaux hiérarchisés (de Marsily G. et Poitrinal D., 1973).

4. LE CALAGE DU MODELE

4.1 Présentation générale des bassins versants

Les cinq bassins versants utilisés sont tous situés sur le Plateau suisse, mis à part l'un d'entre eux(le Necker), qui peut être qualifié de préalpin.

Ces bassins ont fait l'objet d'une étude particulière sur le plan de la qualité de la mesure des débits de basses eaux (Schmidt F., Evard D., Jaton JF., 1983).

D'un point de vue hydrogéologique, ces bassins sont à substratum rocheux faiblement perméable ; ils sont en position fortement drainée et pratiquement dépourvus de gros aquifères quaternaires.

4.2 Les réponses impulsionnelles

Au terme de l'étude de sensibilité des divers paramètres du modèle, les réponses impulsionnelles dégagées sont toutes des fonctions plurimodales. Le premier mode est toujours atteint le premier mois et représente souvent la part principale de la fonction de transfert.

Et chose surprenante, la valeur du premier mode est liéeà la surface du bassin versant par une relation de type puissance. Cette courbe accompagnée des cinq réponses impulsionnelles obtenues est représentée à la fig. 2.

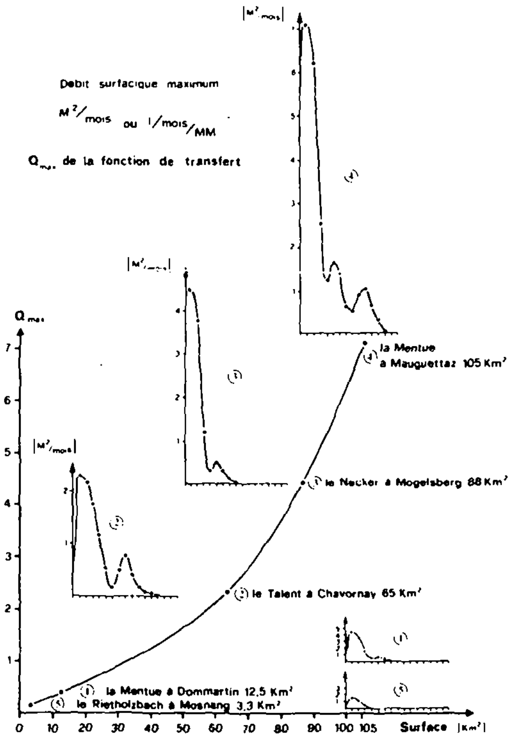

Fig. 2 :Réponses impulsionnelles obtenues et relation de l'ordonnée maximale avec la surface

4.3 Discussion des résultats

Les fonctions trouvées sont donc de type plurimodale. Le premier mode, dont la valeur est liée à la surface, survient lors du premier mois. Après avoir essayé plusieurs longueurs possibles de RI (4-8-16-32), les résultats montrent que l'aspect plurimodal est conservé et que la longueur optimale de la réponse impulsionnelle est de 16 mois sur l'ensemble des bassins testés.

Pour ce qui est des parties terminales de ces RI, deux points de vue successifs peuvent être considérés,à savoir le point de vue mathématique et le point de vue géologique.

D'un point de vue mathématique le problême de la déconvolution étant un problême inverse, il est de ce fait très sensible à de petites erreurs sur les séries d'entrées et de sorties. Il est donc fréquent d'obtenir une réponse impulsionnelle présentant des oscillations,(Thiery D., 1978).

Une étude de sensibilité des différents paramètres de la déconvolution n'a pas permis de mettre en évidence l'unimodalité des fonctions. Par contre, à la fig. 3, l'effet de la suppression d'un mode secondaire sur la qualité de l'estimation par notre modèle est présenté.

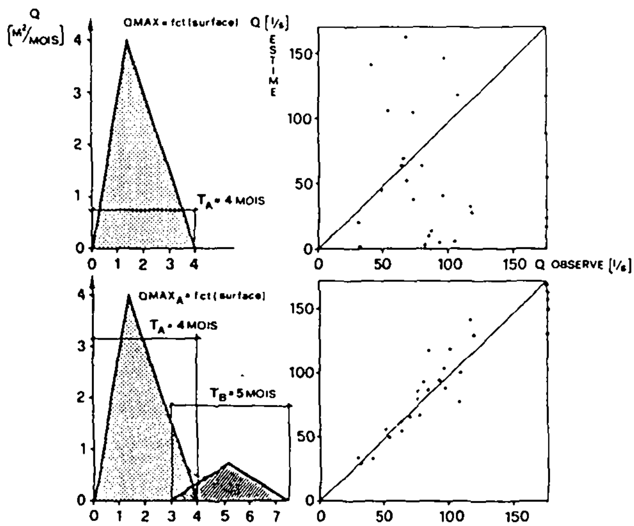

Fig. 3 :Effet de la suppression d'un "pic secondaire" de la RI sur la qualité de l'estimation par convolution

D'un point de vue géologique, l'existence de pics multiples sous-entend la présence de réseaux d'écoulement différenciés au sein de l'aquifère.

A priori une interprétation plurimodale permettrait de lier les différents modes rencontrés aux différents types d'affleurement géologiques recontrés dans le bassin. Cette analogie a été représentée à la fig. 4 pour deux bassins versants emboîtés. Elle permet la mise en évidence d'une hypothèse selon laquelle les différents temps de réaction seraient des constantes pour chaque type de géologie rencontré.

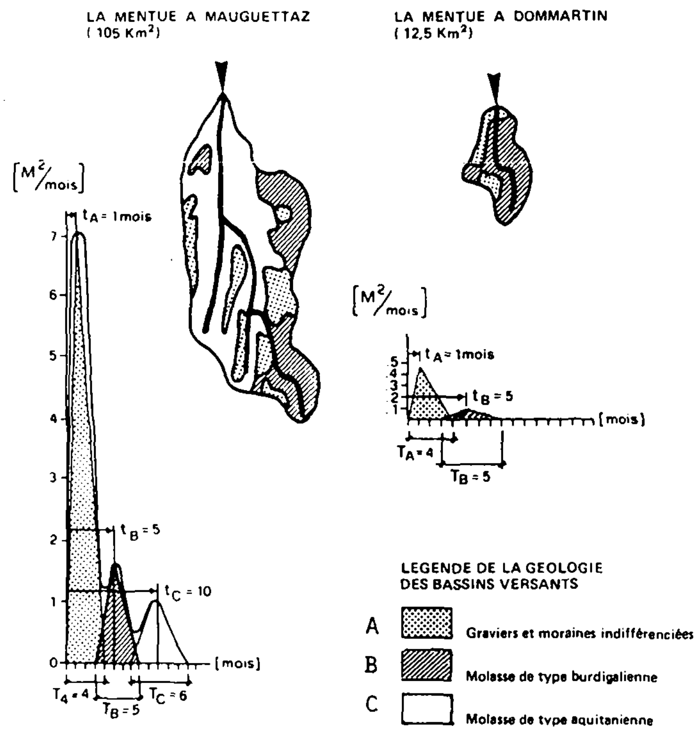

Fig. 4 :Hypothèse pour l'interpretation des pics successifs de la RI

Toutefois, après consultation d'un géologue, cette interprétation plurimodale représente une schématisation n'ayant pas de signification particulière d'un point de vue hydrogéologique.

Pour tenir compte de la partie terminale de cette RI qui semble malgré tout être porteuse d'une information hydrogéologique,des essais comparatifs ont été effectués pour mettre en évidence une éventuelle amélioration de la qualité du modèle par l'utilisation d'une RI unimodale. Mais les résultats des différentes convolutions effectuées restent légèrement en faveur de l'interprétation plurimodale.

Le lissage de la partie oscillatoire de la RI sur une fonction exponentielle et la considération des modes successifs de la RI ont un effet équivalent sur la qualité de l'estimation des débits minimums mensuels.

5. REPONSES IMPULSIONNELLES SYNTHETIQUES

L'interprétation plurimodale présentée dans les paragraphes précédents permet la construction de RI synthétique en considérant des hydrogrammes de types triangulaires spécifiques au type de roche rencontrée dont les paramètres principaux sont l'ordonnée maximale et le temps de base.

5.1 Application unimodale et bimodale

Une application bimodale a été tentée dans le cas de bassins versants présentant au plus deux affleurements géologiques distincts. Ces bassins aptes à la mesure précise des étiages)font partie du même contexte géologique; leurs caractéristiques sont les suivantes:

Tableau 1

Cours d'eau	Station	Types d'affleurement (perméabilité décroissante)	
Le Parimbot	Eschiens	Moraine indifférenciée	
Le Biberenkanal	Kerzers	Moraine indifférenciée	Molasse burdigalienne

Les critères constructifs des réponses impulsionnelles ont été les suivants:

1er mode: L'ordonnée maximale dont la valeur se calcule à l'aide de la surface (voir fig. 2) se situe toujours au premier mois de la réponse impulsionnelle.

Le temps de base correspondant a été fixé à 4 mois. Ce temps correspond à la valeur moyenne observée sur les cinq bassins versants du test (voir fig. 2).

2ème mode: L'ordonnée maximale intervient avec un temps de réponse fonction de la nature géologique de l'aquifère (5 mois dans le cas de la molasse burdigalienne du Biberenkanal à Kerzers).

La valeur de l'ordonnée maximale a dû être déterminée par utilisation d'une série de débits observés. Sa valeur est donc obtenue par calage du modèle. Le caractère "synthétique" de la RI est encore à travailler dans l'état actuel de l'étude.

Le temps de base est aussi lié à la nature géologique (5 mois dans le cas de la molasse burdigalienne).

5.2 Résultats de l'application

La qualité de l'estimation des débits minimums mensuels obtenus peut être visualisée au moyen de courbes chronologiques et corrélatives (voir fig. 5).

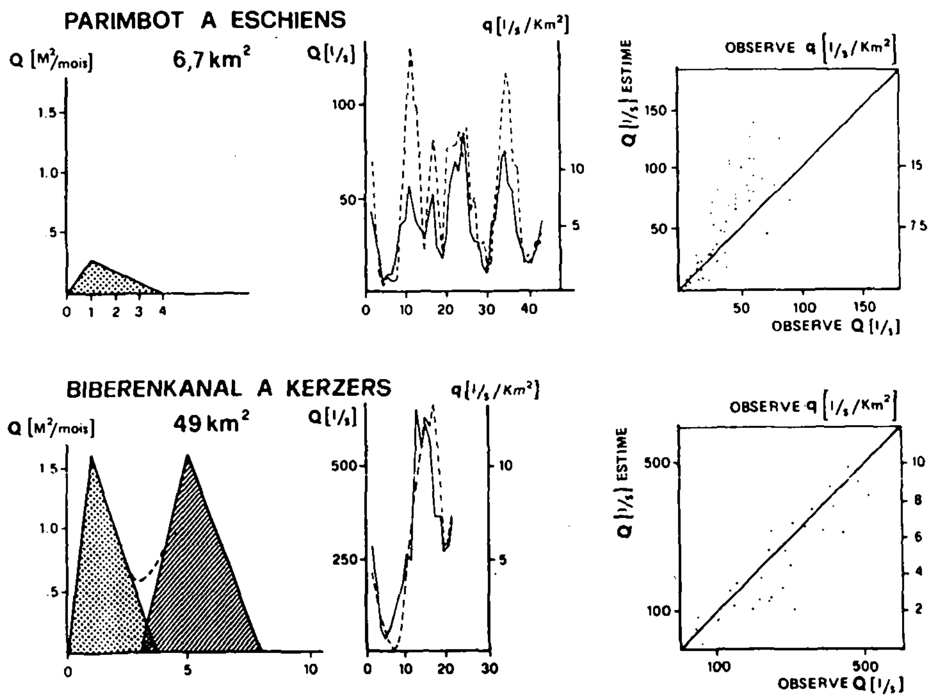

Fig. 5 :Application d'une réponse impulsionnelle synthétique basée sur l'interpretation plurimodale

5.3 Discussion des résultats

Cette approche simple plurimodale permet une estimation satisfaisante des débits minimums mensuels. Elle représente une première mise en valeur de l'importance de la partie terminale des réponses impulsionnelles sur la détermination des débits d'étiage, bien qu'elle soit, dans une première analyse, dépourvue de toute signification physique.

CONCLUSION

Le modèle appliqué se révèle satisfaisant quant à la détermination des débits de basses eaux, caractérisés ici par le débit minimum mensuel.

Toutefois le modèle est peu performant pour la détermination des débits de recharge hivernale car les débits minimums mensuels comportent alors une bonne part de ruissellement. Relevons cependant que le modèle est susceptible d'améliorations en ce qui concerne le choix des entrées et des sorties,d'une part en ce qui concerne l'amélioration de l'explication de la réponse impulsionnelle par la géologie d'autre part. Une telle étude permettrait entre autre l'explication géologique du coefficient de décroissance exponentiel de la RI. Une ébauche de relation a d'ores et déjà été trouvée entre ce coefficient et un indice géologique de perméabilité sur les bassins concernant cette étude.

Ces différents travaux sont programmés dans le cadre du projet du Fonds national suisse d'estimation des débits d'étiage sur des cours d'eau sans mesures directes.

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A NEW COMPROMISE ORIENTED MULTICRITERIA METHOD

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ABSTRACT

In this contribution a new multicriteria method, called "CONSENSUS", is presented. This method suitable to treat both discrete and continuous decision problems, can be seen as an extension of compromise programming or as an extension of Saatys eigenvalue analysis.
The method is very suitable for water resources design problems, since it is oriented towards compromise solutions. By taking explicitly into account the preferences of the different parties concerned with the consequences of the proposed actions, the politically most suitable actions are determined.
After presenting the technical aspects of the method, it is applied to the case study of the water resources planning of the Gete, a river of about 500 km^2 in Belgium. The results obtained with CONSENSUS are compared to the results obtained with ELECTRE I, another well known multicriteria method.

INTRODUCTION

The purpose of this research was to develop a compromise-methodology.

With such a compromise-methodology it should be possible to prepare political water resources decisions, which lead to a co-ordinated and global water management. The methodology should be compromise oriented, so that the proposed solution is the overall most acceptable solution for all important population groups, directly concerned with the water resource management of the studied basis (i.e. the so-called pressure groups).

The development of this compromise-methodology was majorly based on two features:

1. A real-world example, used as a test case;
2. A multicriteria method.

As a test case, the basin of the river Gete was chosen (see fig.1.), since a number of interrelated water quantity and quality problems exists in this basin (see §3. for a description).

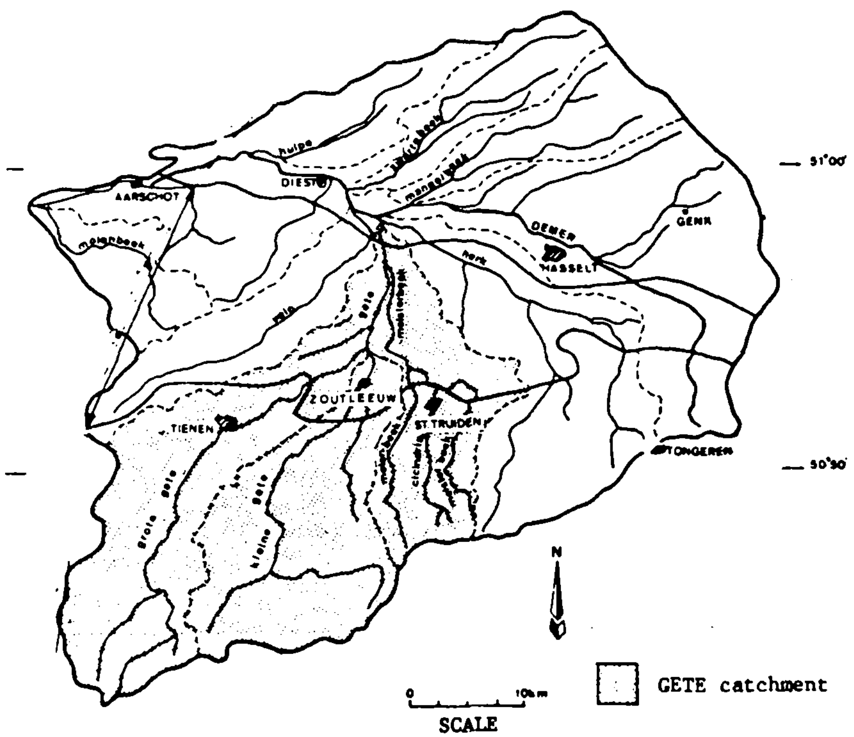

Fig. 1: Catchment of the Demer and the Gete

As multicriteria methods, ELECTRE I (see BENAYOUN et al, 1966) as well as a new method, CONSENSUS, derived from the method of DUJMOVIC (1974 and 1975), were used.

After a short presentation of the CONSENSUS-method in paragraph 2, the alternative plans as well as the criteria, used in the problem of the river Gete, are described in paragraph 3. In paragraph 4 a comparisation is made between the results of ELECTRE I and CONSENSUS, followed by some preliminary conclusions in paragraph 5.

THE CONSENSUS-METHOD

Consider a multiobjective problem with n criteria and m alternative plans or actions. Let c_{ij} be the value of criterion i, for alternative plan j. In the CONSENSUS-method, for each criterion i, a rescaling is done, by means of an uni criterion preference function, $f_i(c_{ij})$, which maps the criterion values c_{ij} into the real values in the interval $[-1,+1]$.

The value $f_i(c_{ij})$ measures the utility or the degree of preference, attached to alternative j, as for as criterion i is concerned. Increasing positive values of $f_i(c_{ij})$ correspond to increasing degrees of aprovement of solution j while decreasing negative values correspond to increasing degrees of disaprovement of solution j. The values +1 and -1 correspond to a saturated level of respectively approval and disapproval. It is the opinion of the author that such saturation levels occur frequently in multicriteria decision problems. For example all solutions where the mean return period of a flood is larger than 100 years, are, in a first approximation, equally acceptable as far as flood protection is concerned. In a problem concerned with the choice of a professional career, all jobs with a monthly net payment higher than say 20000 $ can be equally satisfactory as far as earnings are concerned. Fig.2 gives a possible form of the preference function in this case. The values 100 for the return period and 20000 for the payment are of course personal choices of the decision maker. In fact the complete description of the function f_i is a decision maker's choice (see later on).

Once all uni-criterion preference functions f_i for i=1,...,n are determined, the impact matrix c_{ij} can be converted into a preference matric

$$p_{ij} = f_i(c_{ij}) \text{ for } i=1,\dots,n \text{ and } j=1,\dots,m.$$

Each plan j is now represented by a n-dimensional point, $(p_{1j},p_{2j},\dots,p_{nj})$, in the n-dimensional cube defined by the intervals $[-1,+1]$. Figure 3 gives an example in the case of 2 criteria (n=2). Clearly A is an ideal point while B is an anti-ideal point. The plans 1,2 and 4 clearly are inferior plans, since one or two of the criteria have negative preference values.
Indeed, plans 1 and 4 are dominated by plan 5 and 3, while plan 2 is dominated by plan 3. To make a choice between plans 3 and 5 an overall preference measure of these plans is needed. Clearly such an overall preference measure should be a function of the individual preference values p_{ij} as well as a function of a set of normalized criterium weights λ_i, with

$$\lambda_i \geq 0 \text{ for } i=1,\dots,n \text{ and } \sum_{i=1}^{n} \lambda_i = 1.$$

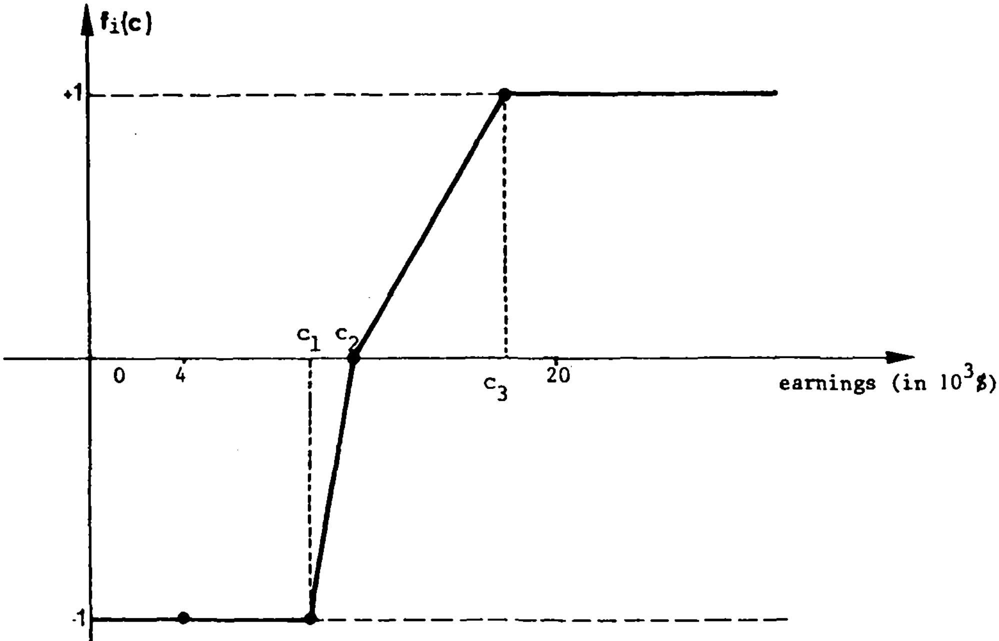

Fig.2: An example of uni-criterion preference function

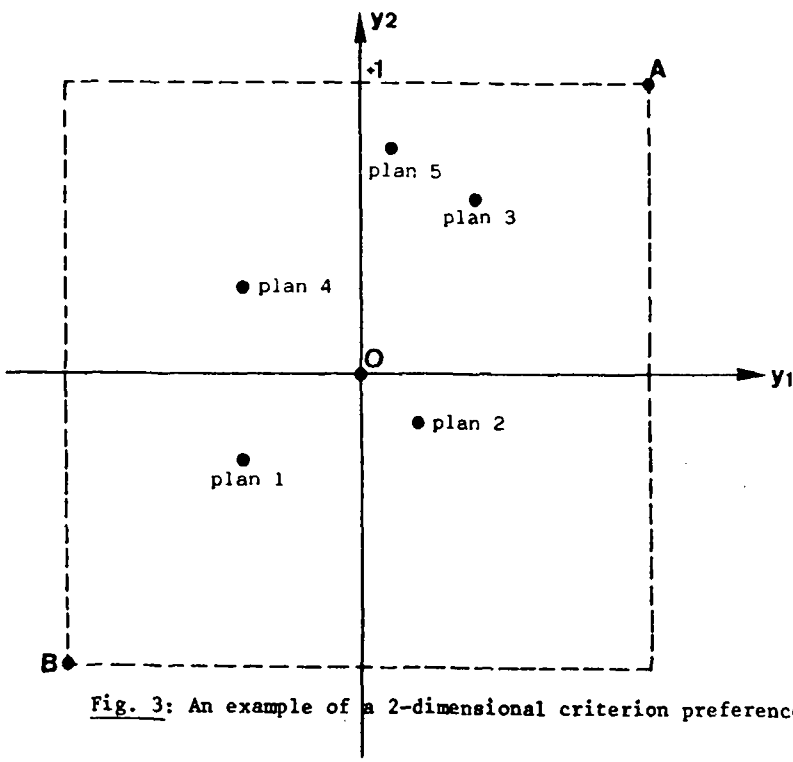

Fig. 3: An example of a 2-dimensional criterion preference space

A first family of overall preference functions is obtained by taking the complement with respect to one, of a generalized distance from the ideal point;

$$f_p(p_{1j},p_{2j},\ldots,p_{nj};\lambda_1,\ldots,\lambda_n) = 1 - \left[\sum_{i=1}^{n} \lambda_i (1-p_i)^p\right]^{1/p}$$

A second family is obtained by considering the distance from the anti-ideal point minus one:

$$f'_p(p_{1j},p_{2j},\ldots,p_{nj};\ \lambda_1,\ldots,\lambda_n) = \left[\sum_{i=1}^{n} \lambda_i (1+p_{ij})^p\right]^{1/p} - 1$$

The parameter p can take any real value larger than 1. These functions have a number of interesting and attractive properties.
One can easily find that

$$\frac{\partial f_p}{\partial p_{ij}} = \lambda_i \left(\frac{1-p_{ij}}{1-f_p}\right)^{p-1} \quad \text{and} \quad \frac{\partial f'_p}{\partial p_{ij}} = \lambda_i \left(\frac{1+p_{ij}}{1+f'_p}\right)^{p-1}$$

so that, for p=1, the function f_p and f'_p are 100% compensatory, in the sense that a decrease of a component i with δ can be offset by an increase of a component j with $\frac{\lambda_i}{\lambda_j}\ \delta$. Hence, apart from a ratio of criterion weights, compensating criterion value changes are equal, independent of these criterion values themselves. This 100% compensatory attitude changes when p increases. The functions f_p then correspond to an increasing concern with the smallest of the p_{ij} values.

In the limit, for $p = +\infty$, one is only concerned with the smallest of the p_{ij} for $i=1,\ldots,n$. It represents the attitude of the teacher in the class who spends all his efforts in improving the knowledge of his worst pupil. The functions f'_p on the other hand correspond to an increasing concern with the largest of the p_{ij}-values when p increases. In the limit, for $p=+\infty$, one is only concerned with the largest of the p_{ij}-values. This corresponds with a teacher's attitude who spends all his efforts on the best pupil of his class and who is absolutely not concerned with all the other pupils.
In summary one can say that:

1° for p=1 the functions f_p and f'_p are 100% compensatory;
2° when p increases, the compensation effect diminishes;
3° the functions f_p model an anti-élite attitude;
4° the functions f'_p model a pro-élite attitude;
5° this pro-or anti-élite attitude increases with increasing p.

The applicability of the consensus-method in the compromise methodology.

It is the author's opinion that the consensus method is particularly suited for the objectives of the compromise methodology as described in the introduction.
The uni criterium preference functions correspond to a reality in the following sense. Representatives of the pressure group who is concerned with criterium i can tell from which level c on they are completely satisfied with the presented solution as far as criterium i is concerned. Hence $f_i(x)$ for $x \geq c$ is equal to +1. In the same way they determine a

value c' which is such that all solutions with a score for criterium i smaller than c' is totally inacceptable. Hence $f_i(x)$ for $x \leq c'$ is equal to -1. In most cases it is also possible to determine a value c" such that $f_i(c")=0$. The value c" should be such that is corresponds to the transition from a negative attitude towards a positive attitude when the value for criterium i is increased from values smaller than c" to values larger than c". The existing state of the river basin usually is a source of inspiration for the determination of c".

The purpose of the compromise methodology is to maximize the political acceptability of the chosen solution by minimizing the global discontentment of the different pressure groups. Hence, it is clear that the global preference functions, f_p, who model an anti-élite attitude, should be used. It is essential to undertake a sensitivity analysis by examining how the solutions change when p varies from 1 to $+\infty$ and when the criterion weights are changed within a reasonable range.

THE ALTERNATIVES AND CRITERIA USED IN THE CASE-STUDY OF THE GETE

General structure of the alternatives

Each alternative plan consists of:
1) one of the 5 flood protection plans F1,F2,...,F5
2) one of the 6 agricultural plans A1,A2,...,A6
3) one of the 4 water quality plans Q1,Q2,...,Q4

The flood protection plans

F1: This is the existing state were a large natural flood plain exists.
F2: In this plan existing river banks are improved to obtain a protection for floods with a return period of 100 years.
F3: In this a completely new river route is built which can pass discharges with a return period of 10 years. This is supplemented by 2 flood-reservoirs with a storage of $\pm$ 4.10^6 m^3.
F4: In this plan only the flood reservoirs are implemented.
F5: Only one flood reservoir is implemented while most existing flood plain areas are preserved.

The agricultural plans

Four possible actions concerning the agricultural destination of the different parts of the river basin are considered: preservation of the existing agricultural use, drainage, extensive agricultural use and land-devaluation. The last two operation modes consist of an ecological minded agricultural use (land-devaluation more than extensivation). In the six agricultural plans these operation modes are applied in some combination in the different area's. Table 1 gives an overview.

Table 1: The six agricultural plan elements

Type of action	Drainable low lands	Non drainable low lands	Drainable high lands	Ecologically valuable high lands
Existing use	A1/A3	A1/A2	A1/A4/A6	A1/A3/A4 A5/A6
Drainage	A2	-	A2/A3/A5	A2
Extensivation	A5/A5/A6	A3	-	-
Land-devaluation	-	A4/A5/A6	-	-

The water quality plans

Q_1: This is the existing state, where all domestic effluent waters are dumped in the river (the industrial development in the basis is very limited).

Q_2: In this plan three conventional water treatment plants are considered to purificate the water of the major city's in the region. All other poluted water is dumped directly into the river.

Q_3: The same three conventional water treatment plants are supplemented by non-conventional purification works such as reed-lands and other ecologically minded actions.

Q_4: The basin is subdevided into 7 purification zones. For each zone a conventional water treatment plant purificates all domestic water.

Description of the considered alternative plans

Table 2 summarizes the 28 plans formulated as combinations of the F,A and Q-plans.

Table 2: The 28 alternative plans

	F_1 A_1	F_2 A_2	F_3 A_2	F_4 A_3	F_1 A_4	F_5 A_5	F_5 A_6
Q_1	P_1	P_2	P_3	P_4	P_5	P_6	P_7
Q_2	P_8	P_9	P_{10}	P_{11}	P_{12}	P_{13}	P_{14}
Q_3	P_{15}	P_{16}	P_{17}	P_{18}	P_{19}	P_{20}	P_{21}
Q_4	P_{22}	P_{23}	P_{24}	P_{25}	P_{26}	P_{27}	P_{28}

The considered criteria

The following criteria were found to be important:

1) Water quality expressed as the total river lenght (in km) were an acceptable water quality exists.
2) Recreation expressed by a dimensionless coefficient expressing the feasibility of the basin for angling.
3) Ecological quality expressed by an ordinal appreciation of the plans given by an ecologist.
4) Agricultural benefits given by the yearly benefits (deficits) caused by drainage (extensivation or land-devaluation) and expressed in 10^6 Belgian franks (BF).

5) Long term emploiment effects. These are due to the agricultural activities as well as to the necessity of maintenance personal for the purification plants (expressed in number of persons per year).
6) Benefit-cost ratio given by the yearly benefits devided by the total yearly cost to develop the considered alternative.
7) Short term emploiment. This criterion measures the direct emploiment associated with building the drainage system and the purification installations (expressed in man-years).
8) Return period of floods in Diest. The flood measures undertaken are part of a flood protection scheme on a larger scale to protect the town Diest (see fig. 1). With this criterion, the influence of the chosen alternative for the Gete, on this scheme is measured. However, due to the flood protection measures taken outside the Gete basin, it turned out that the return period in Diest was larger than 100 years with all alternative Gete plans.

The criterion weights

All criteria which translate the same interests are taken together in one group. Each group is given the same weight of importance. Secondary weights determine the relative importance of the criteria in one group. Table 3 summarizes the situation. It was decided to make a sensitivity analysis by considering all possible combinations of primary weights when these are varied from 1.0 to 1.5 up to 2.0. As such 81 combinations are possible (3^4).

Table 3: Weights of the different criteria

criteria	weights within the group	criteria weights between groups	normalized criteria weights
- water quality	0,7	1,0	7/40
- recreation	0,3		3/40
- nature	1,0	1,0	10/40
- agricultural benefits	0,6	1,0	6/40
- long term emploiment	0,4		4/40
- benefits/costs	0,7	1,0	7/40
- short term emploiment	0,3		3/40

THE COMPARISON OF ELECTRE I AND CONSENSUS

The results of Electre I

Since this method basically works with ordinal scales, all cardinal criteria are treated in the following way: for a criterion i, plan j dominates plan k only if

$$c_{ij} > c_{ij} + \varepsilon_i,$$

where ε_i is an indifference band for criterion i. For ε_i, 5% and 10% of the maximum value of the considered criterion was chosen.
Table 4 gives the results of the analysis for a concordance level δ of 1.33 and an indifference band of 5%. For concordance levels of 1.7 and 2.0 and indifference bands of 5 or 10% simular results were obtained.

table 4: Results of the ELECTRE I method for a concordance level δ of 1.33 and an indifference band of 5%.

plan number	frequences with which these plans are part of the kernel when considering the 81 weight combinations
4	38
15	54
17	81
20	81
22	75
21	9
27	26
others	0

The kernel of the ELECTRE I-method consists of all solutions for which it is impossible to show that their exists a better solution. This means that, when a plan belongs to the kernel, it is not necessary a satisfactory plan. As a consequence, one can not consider the frequency with which a plan belongs to the kernel in a sensitivity analysis on the criterion weights, as a score of that plan. This fact is a major complication in the interpretation of the results. The only definite result is that O_3 seems to be a good plan. The fact that plan 17 has high frequencies means that the ELECTRE I method is not compromise oriented, since 17 is a plan constructed from a purely agricultural viewpoint.

Results of CONSENSUS

The uni criterion preference functions for the 7 criteria were determined and used to calculate the preference matrix. From this matrix the following conclusions were derived. If one eliminates all plans where at least one preference value is equal to -1, only the plans 4, 11, 16, 17, 18, 23, 24 and 25 remain. If one eliminates all plans with preferences values lower than -0.8 only plans 18 and 25 remain.
Further it is clear why plan 17 was frequently in the kernel of the ELECTRE I-method; it scores high for all but one criterion.
Table 5 summarizes the results for the weights of table 3 and for $p=1,2,3,4,5$ and $+\infty$. Only for $p=1$ (100% compensatory attitude) plan 18 is worse than plan 17. For all anti-élite attitudes ($p \geq 2$) plan 18 is the best alternative. Even for $p=1$ there is only a difference in overall preference of 0.2 between plan 18 and plan 17.
The advantage of $p=+\infty$ is that it is an evaluation which is independent of the criterion weights.
The same calculations for other weight combinations showed that the demonstrated results were very stable as far as criterion weights is concerned.

CONCLUSIONS

For the present applications the results of CONSENSUS seem to be far more attractive than those of ELECTRE I. The aspects in which CONSENSUS is an improvement compared to ELECTRE I are:
- the clearness of the results;
- the compromise oriented properties;
- the stability of the results as far as criterion weights is concerned.

Table 5: Results of the CONSENSUS-method for p=1,2,3,4,5 and +∞ and for the normal weights of table 4 (the table contains the numbers of the 10 best plans and their global preference values multiplied by 100)

	order of plans									
	1	2	3	4	5	6	7	8	9	10
p=1	17	24	16	23	10	18	3	9	2	25
	51	49	48	44	30	29	28	23	22	19
p=2	18	17	16	14	23	25	10	3	15	11
	15	06	06	03	02	01	-07	-07	-07	-09
p=3	18	25	17	16	11	4	15	24	23	22
	06	-11	-18	-18	-21	-21	-22	-22	-22	-23
p=4	18	25	11	4	17	16	15	22	24	23
	00	-18	-31	-31	-32	-32	-33	-34	-37	-37
p=5	18	25	11	4	17	16	15	22	8	1
	-04	-24	-38	-38	-41	-41	-43	-43	-46	-46
p=+∞	18	25	4	11	16	17	23	24	1	2
	-33	-57	-80	-80	-87	-87	-93	-93	-99	-99

Since the purpose is to find the "optimal" water resources development plan the ideal would be to study all possible alternatives. In practice it is impossible to study an infinite number of alternatives.
The only possible approach then consists of formulating and evaluating a first set of plans. This stage is then followed by a reformulation stage in which new plans are formulated and evaluated on the basis of the experience gained with the first set of plans.

When such a reformulation was carried out for the Gete problem the following additional advantages of CONSENSUS compared to ELECTRE I became clear. With consensus one has a measure of the improvement obtained by the reformulation, by comparing, for each value of p, the increase of the maximum value of the overall preference function values. With ELECTRE I on the other hand, their is no clear relationship between the kernel of the first set of plans and that of the second set of plans.

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