PROCEEDINGS

Volume 2

VIII IWRA WORLD CONGRESS
ON WATER RESOURCES

SATISFYING FUTURE NATIONAL AND GLOBAL WATER DEMANDS

Cairo, November 21 - 25, 1994

IWRA

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(Volume 2)

INTERNATIONAL WATER RESOURCES ASSOCIATION

NATIONAL WATER RESEARCH CENTER
Ministry of Public Works and Water Resources, Cairo, Egypt

INDEX OF TECHNICAL SESSIONS

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<u>Key</u>
T = Topic
S = Session

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2	045	14:45 - 15:00	Changes in Water Demands in Germany - Challenge and Chance By : Schumann, A.H.
3	158	15:00 - 15:15	Egypt's Efforts Towards Management of Agricultural Water Demand By : Abu-Zeid, M.A.
4	114	15:15 - 15:30	Water Productivity in Egyptian Agriculture: Scenarios for More Sustainable Use By : Kishk, M.A. and J. Lundqvist
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6	030	15:45 - 16:00	Water Resources as a Locational Factor of Human Activities to Satisfy Demands Under Water Drought Conditions, Examples From West Africa By : Lindskog, P.

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2	126	16:45 - 17:00	Optimisation of Water Use in Arid Areas By : Arar, A.
3	068	17:00 - 17:15	Land Reclamation Plans in Egypt and Water Requirements up to Year 2012 By : El-Shibini, F., W. F. Mankarious and M.N. Bayoumi
4	056	17:15 - 17:30	An Attempt to Estimate the Industrial Water Demand By : Benedini, M. and F.M. Spaziani
5	067	17:30 - 17:45	Control of Water Consumption of Rice By : El-Quosy, D.E.
6	168	17:45 - 18:00	Water Demand Management for Navigation Uses Within River Nile By : Saleh, A. and M. Abu-Zeid
7*	087	18:00 - 18:15	Meeting the Water Demands in Egypt: A Future Challenge By : Attia, B., S. Abdel Dayem and F. Attia

<u>**Note**</u>: Papers marked with (*) were not received till the time of issuing this document.

Topic 2: Institutions for Managing Water Demands

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2	028	9:55 - 10:10	Management of Water Demands in Rural Australia By : Pigram, J.
3	052	10:10 - 10:25	Organizations for Irrigation Management By : Ankum, P.
4	131	10:25 - 10:40	The Critical Role of Irrigation " Learning" and Improved System Performance in Meeting Future Water and Food Demands By : Skogerboe, G.V. and D.J. Bandaragoda
5	169	10:40 - 10:55	Organized Water Research in Egypt: The Role of the Academy of Scientfic Research and Technology By : Abu-Zeid, M.A. and G. Abdel-Samie
6	127	10:55 - 11:10	The Role of Wastewater Recycling and Reuse in Water Resources Management Under Conditions of Scarcity By : Shuval, H.

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Coordinator : Shaden Abdel Gawad

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2	122	11:45 - 12:00	The Need for Strategic Research on Water Supply and Demand in Egypt By : Fahmy, H., A.K. Biswas and P.M. Pelletier
3	071	12:00 - 12:15	For An Efficient Management of Water Demand: Farmer Training Service (CES System) in the Lower Seyhan Irrigation Project Area, Adana-Turkiye (A Case Study) By : Tekinel, O. and A.Yazar
4	144	12:15 - 12:30	Towards Integrated Water Management in the Fringes of the Nile Valley By : Attia, F. and H.M. van Leeuwen
5	083	12:30 - 12:45	Technical and Organizational Changes in Irrigation Rehabilitation Programs: Management of the Water Delivery System in Irrigation Improvement Project (Egypt) By : Oad, R.
6	050	12:45 - 13:00	The WUA Approach to Improved System Performance: Experiences from Egypt By : Hvidt, M.

Topic 2: Institutions for Managing Water Demands

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Place : Amneris Salon
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Coordinator : Shaden Abdel Gawad

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3	093	15:00 - 15:15	Regional Integration of Water Demands for Adequate Development of Resources in South Africa By : Barta, B.
4	148	15:15 - 15:30	Institutional Development for Water Demand Management in Semi-Arid Zones-Nigeria's Experience By : Sonuga, J.O.
5	135	15:30 - 15:45	Appropriate Policy Instruments For Managing Water Demands in Nigeria By : Oyebande, L.
6*	051	15:45 - 16:00	Water Resources Management - The Kenyan Dilema By : Ndege, M. M.

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Place : Radames Salon
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Coordinator : Maha Tawfik

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2	032	9:55 - 10:10	Cost Allocation Among Water Service Sectors for Egypt's Nile System By : Elassiouti, I. M. and A.S. El - Zaher
3	005	10:10 -10:25	Extended Economical Standpoint on Flood Control Works By : Botzan, D.A. and T. Botzan
4	091	10:25 - 10:40	Medusa Bag Projects for the Ocean Transport of Freshwater in the Mediterranean and Middle East By : Cran, J. A.
5	059	10:40 - 10:55	A Plan for Cost-Recovery of Water Facilities in the Irriagtion System By : El-Shibini, F.
6	060	10:55 - 11:10	Cost-Recovery for Demand Management of Water: Economic Perspective in Egypt By : Nasr, M. L.

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Session 1

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Place : Eugenie Lounge
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Coordinator : Hossam Fahmy

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2	017	9:55 - 10:10	Integrating Socio-Economic and Environmental Aspects in Water Resources Planning and Management By : Mohorjy, A. M.
3	086	10:10 - 10:25	Drainage Water Quality and Environmental Issues By : Abdel-Gawad, S.T.
4	075	10:25 - 10:40	Valorisation des Eaux Usées Dans Sud Tunisien By : Makni, H., F. Brissaud, S. Tajina and H. Ben Dhia
5	095	10:40 - 10:55	Effect of Waste Discharges on the Water Quality of Bahr El-Baqar Drain System, Eastern Nile Delta By : El - Sayed, A., S.T. Abdel Gawad, S. M. Abdel Gawad and K. Ibrahim
6	149	10:55 - 11:10	Assessing Performance of Irrigation and Drainage; Examples From Egypt By : Bos, M.G., S. Abdel Dayem and F.A. Attia

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Date : Thursday Nov. 24
Place : Eugenie Lounge
Time : 11:30 -13:00

Coordinator : Hossam Fahmy

1	039	11:30 - 11:45	Socio - Economic and Environmental Impact of Coastal Processes on Future Water Demand in Egypt By : Khafagy, A.A., A.M. Fanos, M.G. Naffaa and M.N. Anwar
2	049	11:45 - 12:00	Potential Climate Change Impacts on Nile Basin By : Saleh, M., K. Strzepek and D. Yates
3	166	12:00 - 12:15	Fertilizer Abuse as a Possible Threat to Water Quality in Egypt By : Abdel Monem, M., M.A. Abdel Khalek and M.B Abdel Ghani
4	110	12:15 - 12:30	Environmental Impact of Drainage System on Lake Qarun in Fayoum By : El-Atfy H. and M. Eissa
5	065	12:30 - 12:45	Matching Evolving Water Quality Management Strategies to Monitoring System Design By : Soliman, W. R. and R. C. Ward
6	046	12:45 - 13:00	Dealing with Water Demand Margins By : de Haas, A. W.

Topic 4: Environmental Aspects of Demand Management

Session 3

Date : Thursday Nov. 24
Place : Eugenie Lounge
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Coordinator : Hossam Fahmy

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2	035	14:45 - 15:00	Limits to Hydropower Development in the Himalayan Rivers of Nepal By : Shrestha, D.L. and P. K. Mool
3	013	15:00 - 15:15	Improving the Hydrologic Cycle and Water Environment in River Basins Undergoing Urbanisation By : Takahashi, W. and K. Kamitakahara
4	150	15:15 - 15:30	Groundwater Management With Different Irrigation Demand Alternatives By : Dabbagh, A.E. and W. A. Abderrahman
5*	161	15:30 - 15:45	Impact of Development Irrigation in Central Asia on the Problem of Aral Sea and Ways of Solution By : Dukhovny, V.

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2	029	9:55 - 10:10	Non-Conventional Water Resources Development Alternatives to Satisfy the Water Demand in the 21st Century By : Murakami, M. and K. Musiake
3	159	10:10- 10:25	Satellite Based Distributed Operational Hydrometeorological Forecast System for the Nile River By : Attia, B., M. Andjelic and C.B. Barrett
4	124	10:25 - 10:40	Study of Hydrological Situation in Gaza Strip By : Abu - Mailah, Y.
5	140	10:40 - 10:55	Flow Forecasting for Flood Control and Water Management. By : Ogink, H. J. M., R.A.H.A. Thabet and M.E. Nur
6	054	10:55 - 11:10	Water Resources Development in Gash Delta , Sudan By : Ahmed, A.A.

Topic 5: Demand Management for International Water Bodies

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2	121	11:45 - 12:00	Water Demand Management in Canada - Past, Present, and Future By : Tate, D. M. and A.M. Kassem
3	164	12:00 - 12:15	Water Resource Management in the Mediterranean Basin By : Hamdy, A., M. Abu - Zeid and C. Lacirignola
4	136	12:15 - 12:30	Stochastic Modeling of the Nile River Flows By : Salas, J.D., M.W. Abdel Mohsen and B.B. Attia
5	069	12:30 - 12:45	Southeastern Anatolia Project (GAP) of Turkey: Water Resources Development Within the Context of Integrated Regional Socio Economic Development. By : Unver, O.
6	109	12:45 - 13:00	Water Resources and Water Demand Assessment in the Salawin River Basin in Thailand By : Onta, P.R., R. Loof, A. Das Gupta and R. Harboe
7	009	13:00 - 13:15	Techno-Political Decision-Making for Water Resources Development: The Jordan River Watershed By : Wolf, A.T. and M. Murakami

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3	141	15:00 - 15:15	Strategic Water Planning , An Exercise Case Study: Egypt. By : Khouzam, R.F.
4	147	15:15 - 15:30	A Model for Effective Irrigation Demand Reduction and Water Shortage Management By : Abderrahman, W. A.
5	058	15:30 - 15:45	Evaluation of Optimal Stochastic Operational Policies for the High Aswan Dam By : Abdelkader, H., J.W. Labadie and D.G. Fontane
6	084	15:45 - 16:00	Fast Basin Model for Management of the Resrvoirs on the Seine River and Tributaries By : Morel - Seytoux, H. J.
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Coordinator : Khaled Abu -Zeid

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3	063	10:10 - 10:25	Modeling Irrigation and Drainage Management in the Presence of Water Table By : El - Hessy, F.
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5	108	10:40 - 10:55	Optimal Conjunctive Use of Surface and Groundwater Resources in Egypt By : El-Beshri, M.Z. and J.W. Labadie
6	165	10:55 - 11:10	Multi Purpose Irrigation Canals - Generation of Micro Hydro Electric Power By : Karam, G. E. and I. Najjar

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4	099	14:45 - 15:00	Representative Rainfall Pattern Suitable for Hydrologic Design in Sinai By : Zaki, A.F., M.Sh. Elmanadeli, M.F. Helwa and H.A. Ibrahim
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6	103	15:15 - 15:30	Planification D'un Réseau Météoroloique Application a la Mauritanie By : Ould Moine, S.M., J. Llamas and G. Morin
7	098	15:30 - 15:45	Simulation of Bed Changes and Flow Conditions Due to Suspended Sediment at River / Reservoir System "4 - D Sidhom Model" By : Sidhom, M. and G. S. Ebaid

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Coordinator : Hossam Fahmy

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2	162	9:55 - 10:10	Meeting Future Water Demands in Palestine through Population Redistribution By : Haddad, M.
3	145	10:10 - 10:25	Rainfall - Runoff Water Harvesting Prospects for Greater Amman and Jordan By : Preul, H. C.
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Coordinator : Hossam Fahmy

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FLOW FORECASTING FOR FLOOD CONTROL AND WATER MANAGEMENT

H.J.M. Ogink(1); R.A.H.A. Thabet(1); Dr. Mohamed El Amin M. Nur(2)

ABSTRACT

In 1991-92, a flood early warning system (FEWS) was established for the Blue Nile and Atbara River in The Sudan, with rainfall as its starting point. A Flood Early Warning Centre was established at the Ministry of Irrigation and Water Resources in Khartoum, receiving real-time data from METEOSAT as well as from monitoring points along the river, and producing flood forecasts that cover the reach from the border with Ethiopia down to Dongola, some 320 Km south of the border with Egypt.
Besides flood forecasting, the FEWS has also proven to be an efficient and reliable system for water management, especially for the day to day reservoir management. Various reservoir operation strategies were investigated, which demonstrated the usefulness of FEWS for flood management. In addition, the system provides necessary information to improve other water management aspects, such as storage for irrigation and hydropower generation, as well as reservoir sedimentation.

SOMMAIRE

LA PRÉDICTION DE DÉBIT POUR CONTRÔLE DES CRUES ET LA GESTION D'EAU

En 1991-'92 un système d'avertissement prompt de crue (SAPC) a été établi pour le Nil bleu et la fleuve Atbara dans le Soudan, qui a comme point de départ la régime de pluie.
Un Centre d'Avertissement Prompt de Crue a été fondé au Ministère de l'Irrigation et des Ressources d'Eau á Khartoum, qui recoit directement les données du METÉOSTAT ainsi que des points de mesure le long du fleuve. Ils donnent les prévisions de crue sur la partie du fleuve de la frontière avec l'Éthiopie jusqu'a Dongola, quelque 320 km au sud de la frontière Égyptienne.
A part des prédictions de crue, le SAPC a aussi prouvé être un système efficace et sûr pour la gestion d'eau, spécialement pour la gestion de jour après jour des réservoirs. On a étudiées des différentes stratégies à gérer les réservoirs, ce qui a démontré l'utilité du SAPC. En plus, le système donne de l'information nécessaire à améliorer des autres aspects de la gestion d'eau comme le stockage d'eau pour l'irrigation et pour la production de l'énergie hydro-électrique ainsi que pour la sédimentation dans les réservoirs.

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1. INTRODUCTION

In the wake of the devastating flood of 1988, which caused considerable damage in the Khartoum area and northern Sudan, a comprehensive reconstruction programme was launched and financed by the World Bank. Besides repair of damages and rehabilitation of infrastructure, the reconstruction programme comprised components that would enable Sudan to cope with future floods: *the establishment of a flood early warning system.* To ensure its effectiveness, the lead time for forecasting of (dangerous) floods should be as long as possible. In view of the proximity of the watershed in Ethiopia and the short travel time of the flood wave to Khartoum, this inevitably means that the system must have rainfall as its starting point; *real-time* rainfall, supplemented - if possible - with a *forecast* of expected rainfall.

In just little more than one year, such a system was developed, set up and verified by DELFT HYDRAULICS and the TAMSAT Group of the University of Reading. At the Ministry of Irrigation and Water Resources (MOIWR) in Khartoum a Flood Early Warning Centre was established, receiving real-time data from METEOSAT as well as from monitoring stations along the river, and producing flood forecasts that cover the rivers between the borders with Ethiopia and Egypt.

Right from the beginning of its testing, during its maiden 1992 flood season, the **Flood Early Warning System (FEWS)** produced satisfactory results. With the introduction of some improvements, the results in the next - and higher - flood season 1993 were even better.
In addition to flood forecasting, the FEWS has also proven to be an efficient and reliable system for flood management as well as water management, especially for the *day to day reservoir management.*

The Flood Early Warning System (FEWS) as implemented in the Sudan is described in Section 2 and its performance in the past two flood seasons is dealt with in Section 3. Based on a re-evaluation of the various FEWS components during the past flood season potential further improvements are discussed. The potential of the system as a flood control and water management tool is further dealt with in Section 4. Finally some aspects of the sustainability of the FEWS are discussed in Section 5.

2. NILE FLOOD EARLY WARNING SYSTEM

The objective of the developmemt of the Nile Flood Early Warning System (FEWS) was to enable a more timely warning for floods on the Nile in the Sudan and to provide an operational tool for flood management, see Grijsen et al (1992), El Amin El Nur et al (1993).

Nile floods in the Sudan are solely generated by rainfall over the upper catchments of the Blue Nile and Atbara Rivers in Ethiopia and Eritrea. Rainfall in The Sudan hardly contributes to the Nile flow. The lead times between rainfall and the river flow at various spots along the Nile river is shown in Figure 1. It shows that by accounting for the rainfall-runoff process in the upper catchments extra gains in lead time of **one** to **two days** (Atbara-Setit) and of **three days** (Blue Nile) are obtained relative to river flow forecasting starting off from the border stations. Therefore, the Nile FEWS covers the Blue Nile and Atbara basins fully The White Nile, although contributing little to the Nile floods, is included in the FEWS for its lower part downstream of Malakal in order to incorporate possible water management options through the Jebel Aulia reservoir. The FEWS extends as far as Dongola on the Main Nile.

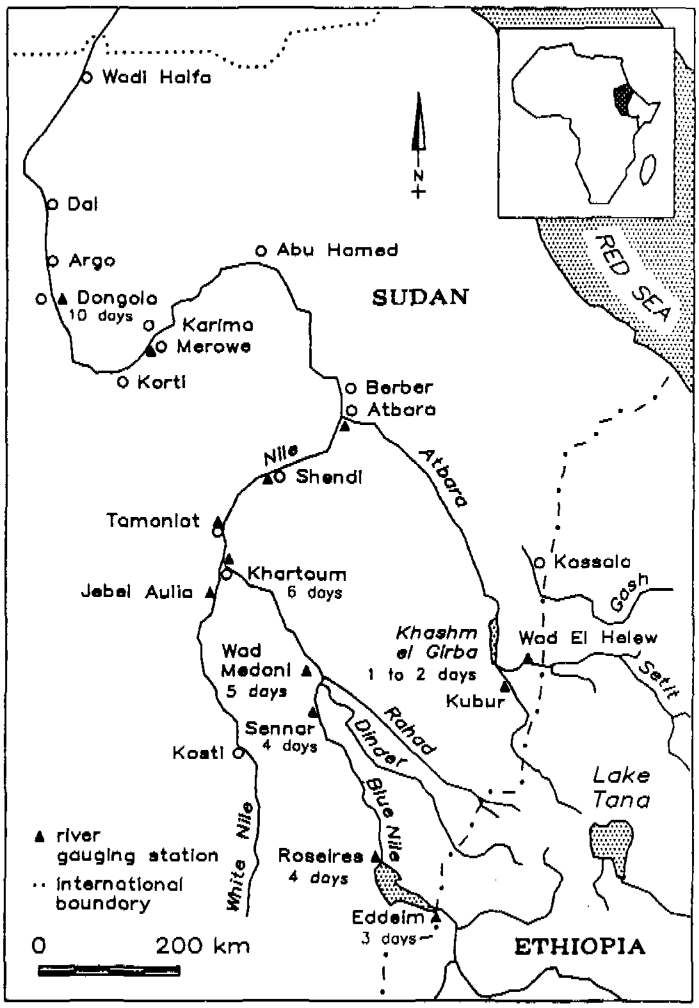

Figure 1 Area covered by the FEWS with lead times

The FEWS consists of three main components, see Figure 2:

1. a Primary Data User Station (PDUS) with relevant software for receiving and processing METEOSAT thermal infra-red (TIR) images on a half-hourly basis (AUTOSAT/-ARCS). These data are used for the estimation of daily rainfall quantities from cold cloud duration and coverage data over the catchments of the Blue Nile and Atbara rivers.
2. a communication system for real-time transmission of water levels in the Blue Nile, Atbara River and Main Nile in The Sudan to the Flood Warning Centre in Khartoum.
3. a computerized Flood Forecasting System, consisting of rainfall-runoff and flow routing models and a temporary database with an appropriate userinterface, allowing smooth and rapid data processing and forecasting.

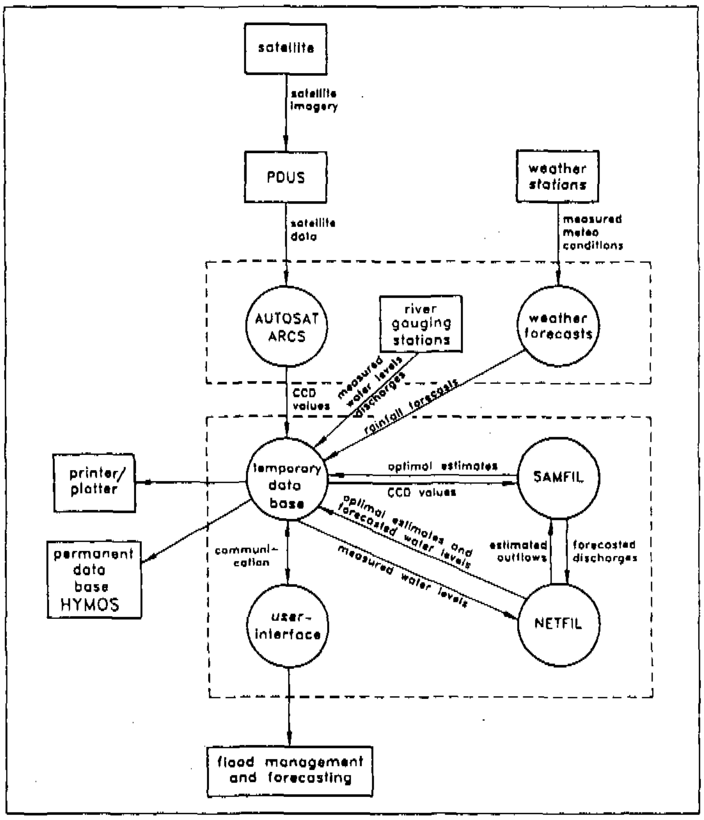

Figure 2 Structure of the FEWS

In the absence of real-time information on rainfall over the Ethiopian catchments METEOSAT TIR images are used to derive daily total Cold Cloud Duration (CCD) and Coverage (CCC) data for the various catchments as a basis for rainfall estimation. The method, developed by the TAMSAT group of the University of Reading (UK), is based on the assumption that rainfall is associated with cloud tops colder than a predetermined temperature threshold, such as -50 degrees Celsius. Linear regression analysis has been applied to establish relationships between rainfall quantity and the CCD and CCC values. The relations differ from catchment to catchment and with the time of the year.

The basis for the mathematical models in the FEWS forms DELFT HYDRAULICS' SAMFIL rainfall-runoff (based on the well known Sacramento model) and NETFIL flow routing models respectively. The upper parts of the Blue Nile, Atbara and Setit rivers are covered by the rainfall-runoff models, which produce inflow to the river system at the Ethiopian-Sudanese borders. From there onward the river routing model takes over to route the flow through the main river system with reservoirs. The models are equipped with a data assimilation algorithm (Kalman Filter), which combines calculated results on water levels and flows with actual observations, received daily via the HF-radio communication system. This allows the state of the models to be updated daily in line with the actual observed state of the rivers, thus creating the best possible starting-point for the next forecasting run. Reservoir operation rules, irrigation off-takes etc. are taken into account.

Each day during the flood season forecasts are made for a period of ten days. This is approximately the lead time between rainfall events over Ethiopia and the rise in water level at Dongola. Flood management strategies can be included by making additional forecasting runs with modified operations of reservoirs.

The forecasting ten days ahead requires estimates of the rainfall in the days to come. At present a wet, average or dry rainfall scenario is used dependent on the weather forecasts by the Meteorological Department in Khartoum. These scenarios have been derived from point-rainfall data in recent years.

3. FEWS PERFORMANCE AND POSSIBLE IMPROVEMENTS

The FEWS started its operation in the flood season of 1992, so experience is now available for two flood seasons.

The quality of the forecasts in the lower part of the system even several days ahead is very satisfactory as a result of the excellent performance of the flow routing part of the system. In the last flood season, after implementation of adaptations to the FEWS, the error in the water level forecasts at Khartoum and Shendi 3 days ahead was only 0.05 to 0.15 m respectively, whereas 5 to 6 days ahead the errors were still less than 0.15 to 0.20 m.

Further upstream the differences between measurement and forecast grew somewhat due to less accurate inflow forecasts produced by the rainfall-runoff model. Basically four factors are accountable for this:

1. Inaccuracies in the rainfall estimates because differences in local meteorological conditions appear to affect the rainfall-CCD relation.
2. Limited quality of short and medium range rainfall forecasts.
3. Doubtful quality of evaporation estimates. Evaporation is an important component in the catchment water balances, but at present insufficient climatological and vegetation data from the Ethiopian part of the Blue Nile, Atbara and Setit basins are available for accurate estimates.
4. The absence of any runoff data from inside the upper catchments, and hence lacking knowledge on their drainage characteristics have led to a strong lumping in the rainfall-runoff modelling for these catchments. Due to this any physical interpretation

to the model parameters has become meaningless: the models have become black boxes. Sofar, a more distributed approach, to account better for the spatial variation in the rainfall alone, but without any knowledge about the drainage characteristics of the sub-catchments, did not prove to be effective.

An important improvement in the filter performance of the rainfall-runoff model was obtained by adjustment of the Kalman Filter noise parameter values of the various conceptual reservoirs in the model. The effect of the adjustment is shown in Figure 3 for the 1992 flood season; the errors in the runoff reduced from 5% to 1.7%.

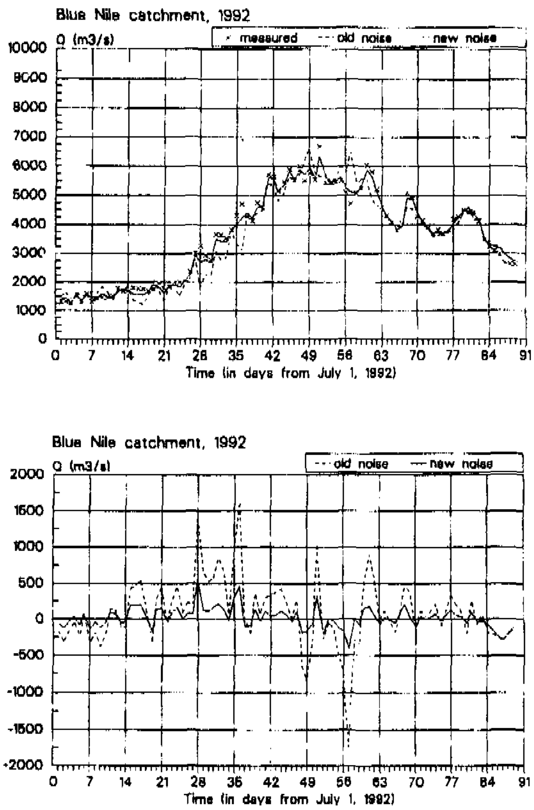

Figure 3 Effect of noise parameters on performance of filter in rainfall-runoff model

Studies carried out during the 1993 flood season have led to the conclusion that the performance of the rainfall-runoff modelling part of the FEWS can substantially be further improved. Some possible improvements are indicated below:

- Segmentation of the Blue Nile catchment in North-East and South-West regions to account for differences in the rainfall-CCD relation. Further improvement can be obtained by a real-time calibration of the rainfall-CCD relation, requiring data from Ethiopia.
- Investigations revealed that at least the area around Lake Tana should be separated from the rest to give full account for the spatial variability in the rainfall, also in view of longer travel times to the Sudanese border relative to the remainder of the catchment.
- Introduction of rainfall forecasts from the European Centre for Medium Range

Weather Forecasts to replace the wet, medium and dry scenarios would improve the forecasts further ahead.

- Collection of reliable climatological data of the Blue Nile, Atbara and Setit basins to get unbiassed historical evaporation estimates. With these estimates a recalibration of the rainfall-runoff models should be carried out.
- In combination herewith the FEWS algorithms should be extended to incorporate real-time satellite based data on absorbed solar energy and vegetative cover for daily adjustment of the potential evaporation estimate relative to its long term mean monthly values.
- Collect a consistent set of data on topography, soils, vegetation, rainfall, evaporation, water levels and discharges of the upper catchments to allow for a segmentation and recalibration of the rainfall-runoff models.
- Collection of real-time water level data from Ethiopia.

These recommendations imply that a good co-operation between the Ethiopian and Sudanese water resources authorities to establish a free exchange of meteorological, climatological and hydrological data between Ethiopia and the The Sudan would be necessary. Indications are that discussions in that direction have recently started, and its importance cannot be stressed enough!

The FEWS could also benefit from the activities under the Egyptian Nile Forecast System Project, which is a joint activity of NOAA and the Ministry of Public Works and Water Resources in Cairo. A close collaboration would be most valuable:

- to execute joint studies on rainfall and evaporation estimation and on rainfall-runoff modelling;
- to exchange data and information on catchment layout and topography, river cross-sections, hydrometry, hydrology, etc.

Joint studies on rainfall-runoff modelling would be of much interest as for the Egyptian Nile Forecast System a fully grid-based approach up to satellite pixel level is used in contrast to the strong lumped approach practised in the Sudan FEWS. If more data become available on the basin characteristics, the distributed approach may give directives as to the admissable degree of catchment lumping.

4. FLOOD AND WATER MANAGEMENT

The existance of the Roseires and Sennar reservoirs on the Blue Nile, the Khashm El-Girba reservoir on the Atbara river as well as the Jebel Aulia reservoir on the White Nile allows to some extent the implementation of flood management strategies to mitigate inundations. It also allows for improved water management for irrigation and hydropower generation purposes.

At present a discharge dependent seasonal strategy for the operation of the reservoirs on the Blue Nile and Atbara is in use, to minimize sedimentation in the reservoirs and to store sufficient water for irrigation and hydropower production. It roughly implies that after an initial filling early July only water in the tail of the hydrograph as from early September onward is stored. The heavy sediment loaden part of the hydrograph in the first half of the flood season is discharged freely to keep the flow velocities in the reservoirs high in order to avoid sedimentation. Severe sedimentation has taken place in the past. Within 20 years the Roseires reservoir capacity was reduced by 30%, whereas from the Khashm El-Girba reservoir only some 40% of the original capacity is left after 30 years of operation. In the absence of White Nile floods a different strategy is being used for Jebel Aulia reservoir. Here a two steps filling strategy takes place from mid July till early October, interrupted at the end of August to allow the build up of storage when the levels near Khartoum become high. The main objective of the dam is to head up water for the pumping schemes along the White Nile and for navigation.

Investigations were carried out of alternative reservoir operation strategies for flood management illustrated by their implementation for the severe Blue Nile flood of 1988. Analysis showed that a substantial reduction in flooding of the areas around Khartoum could have been obtained by reducing the maximum release from Roseires reservoir to (a) 7000 m^3/s, assuming little contributions from the tributaries Rahad and Dinder d/s of Roseires. However, in 1988 the tributaries appeared to have contributed considerably, so a second strategy (b) reducing the release from Roseires reservoir to (b) 6800 m^3/s was applied as well. A third strategy (c) included reduced releases from Jebel Aulia reservoir on the White Nile during the passage of the flood wave from the Blue Nile to reduce the levels near Khartoum. If sufficient storage capacity in Jebel Aulia reservoir is not available, storage can be generated by releasing extra water before the passage of the flood wave from the Blue Nile. In the used strategy the release from Jebel Aulia reservoir was reduced to 50 m^3/s for a period of 35 days. Finally, (d) a combination of the strategies (a) and (c) was investigated. The efficacy of the flood management strategies on the levels in Khartoum is shown in Table 1 and Figure 4.

Calculation	Reduction maximum levels at Khartoum (m)
a) Maximum release Roseires 7000 m^3/s	0.33
b) Maximum release Roseires 6800 m^3/s	0.41
c) Release Jebel Aulia reduced to 50 m^3/s	0.21
d) Combination a) and c)	0.35

Table 1 Efficacy of flood management strategies

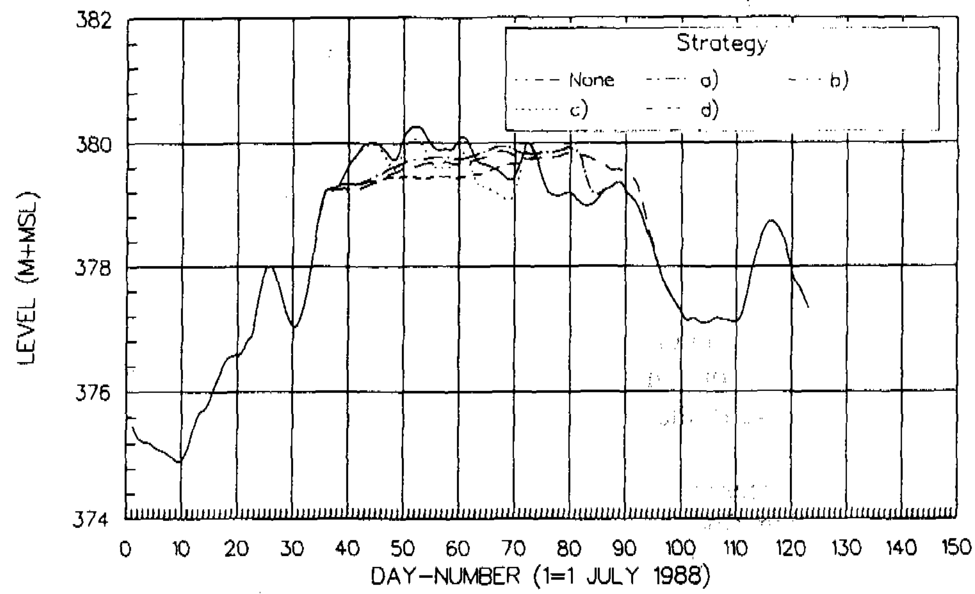

Figure 4 1988 flood levels at Khartoum under various flood management strategies

Table 1 and Figure 4 reveal that storage of flood water in the Roseires reservoir would reduce the flood levels near and downstream of Khartoum considerably, but at the cost of additional sedimentation in the reservoir. For scenario (a) during a period of 46 days flood water is stored whereas for strategy (b) the period of storage is extended to 56 days. Reservoir sedimentation computations carried out with DELFT HYDRAULICS' program SERES for the Roseires reservoir indicated extra sedimentation due to the strategies (a) and (b) respectively of about 75 and 100 Mm^3 relative to an initial storage capacity of 3000 Mm^3. The above figures are some three to four times the annual sedimentation amounts under the present strategy. The extra sedimentation may seem high but one has to take into account that such a strategy is only seldom necessary; the flood of 1988 was very extreme. The economical consequences of such strategies should therefore be further investigated. The value of the FEWS for implementation of one of the strategies is the forecasting of extreme flows from the Blue Nile and of the tributaries and the effects of the flows on the water levels further downstream up to ten days ahead, which gives sufficient time to decide on the implementation of a specific strategy.

The table and figure also reveal that a reduction of 2 dm can be obtained by reducing the realease from Jebel Aulia (strategy (c)), without adverse effects for reservoir sedimentation. It is clear that ample storage capacity should be available in the reservoir to enable temporary storage of White Nile flow. This storage can be created by:

- maintenance of low water levels during the Blue Nile flood season, which will have negative consequences for the upstream pumping schemes;
- extra release from Jebel Aulia reservoir as soon as the FEWS forecasts high Blue Nile discharges. In the ideal situation a period of 6 days is available (lead time) to create extra storage in the reservoir.

Further reduction of flood levels near Khartoum could be obtained by allowing reverse flow at Jebel Aulia dam, in order to store temporarily Blue Nile water in the reservoir. However, at present reverse flow at Jebel Aulia is not allowed for stability reasons, although during the first years of operation reverse flow was a common practice. Based on the efficacy of strategies including Jebel Aulia reservoir a further investigation of the potential of this option is strongly recommended.

Besides flood management, the FEWS provides also useful information for water management, hydropower generation and storage for irrigation, as the FEWS allows the operators to look several days ahead. This is particularly advantageous for the short term planning of releases for irrigation, say at weekly or ten daily basis, since the FEWS provides the best estimate possible about the net inflows to the reservoirs at such a time base.

Based on the previous discussion of possible flood and water management strategies an economic analysis including flood damage, reservoir sedimentation and effects on hydropower production and irrigation schemes is recommended to enable an optimal choice of future operation strategies. Needless to say that such investigations should include also reservoir sedimentation computations.

5. SUSTAINABILITY OF THE FEWS

Training in the operation of the FEWS and in the theoretical background of its components is a prerequisite to ensure a sustainable operation of the FEWS in the future. Several training courses were held ranging from dedicated courses concerning rainfall estimation for hydrological purposes and courses on the Flood Early Warning System to extensive on-the-job training in the field and in the FEWS Centre.

Furthermore, much attention was devoted to the set up of a reliable database for hydro-meteorological data. Such a database is necessary to be able to update regularly the relations for the external and internal hydrological boundary conditions regularly. Automatic procedures have been developed under the FEWS to transfer regularly data from the temporary database under FEWS to the permanent Water Resources database in the Data Centre of the Ministry of Irrigation and Water Resources in Khartoum. The latter is based DELFT HYDRAULICS' database management and processing system HYMOS.

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WATER RESOURCES DEVELOPMENT IN GASH DELTA, SUDAN

Abdalla Abdelsalam Ahmed[1]

ABSTRACT

In this paper the present irrigation system in the Gash Delta is evaluated. New design approach to utilize efficiently the available water resources is investigated. This is done through integrated exploitation of the surface, groundwater, and rainfall, aiming to increase the irrigated area and crops production. The paper also seeks ways and means to reduce the negative effects of the high sediment load on the irrigation system. Siting of the canal offtake on the outer bend of the river is found to be the most effective method to increase the water supply and reduce the sediment intake at the same time. The water distribution at the field level is changed in such away to reduce the water losses both through evaporation and percolation. The high water holding capacity of the Gash soil is utilized to grow selective crops.

LE DEVELOPPMENT DES RESOURCES DE L'EAU AU DELTA DE GASH, AU SOUDAN

EXTRAIT

Dans ce dossier, on a évalué l'actuel systéme irrigable au Delta de Gash. Ici on propose une nouvelle vision du nouveau dessein al 'utilisation des resources d'eau disponible. Alors ce dessein est fait, avec le but, à travers l'exploitation integrale des surfaces, l'eau du sol et la cote de pluviosité, d'augmenter des surfaces irriguees et la production des récoltes. Ce dessein cherche aussi à trouver les facons et les moyens dont on peut reduire les effets négatifs de grand faix d'alluvionnement sur le systéme irrigable. On á trouvé qu'au même temps, le meilleur mode efficient pour augmenter l'approvisionnement de l'eau et réduire le sédiment intérieur est en créant le canal sortant du fleuve à la courbe extérieur du fleuve. De même facon, la distribution de l'eau au niveau des fermes est completement changee en vue de reduire de l'eau manque par l'évaporation ou le filtrage. La grande capacité de l'eau dans le sol est alors utilisée pour qu'on puisse cultiver des récoltes selectionnées.

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1. INTRODUCTION

The water resources development in the Gash area in the Eastern part of Sudan represents quite a challenge to the scientists and engineers in charge. The area is wide and fertile, while the available water is seasonal and limited. The problem can be alleviated if the groundwater and rainfall are used in integrated manner with surface water from river Gash. The development and rehabilitation of the scheme is highly dependent on this idea i.e. integrated water resources use.

River Gash which is mainly responsible of forming the delta (approximately 300,000 ha), Figure (1), is originating from Eriterea highlands and partially from Ethiopian Plateau. The Delta is formed by successive spates of the river Gash, therefore, it is considered as one of the richest soil in the world.

According to the available literature the irrigation has been practiced in the Delta since the last century. Spate irrigation in particular has started in 1905 and continued to grow until finally an area of nearly 140,000 ha was brought under the command of the present irrigation system.

Over the last 20 years the old irrigation system has continuously deteriorated so that only 15,000 to 20,000 ha has been managed to be flooded.

One of the major problems, which has serious impact on the irrigation, is the obtaining of reliable canal offtakes from such an unstable river. The fact that along its course (approximately 120 kms), the river runs to the Gash Dia (flat area) without any controlling structure, makes it more complicated. It often turned out that a newly built offtake did not work well, while those which had been working for years stopped, as the river changed its course. Difficulties occur because of changes in the river itself. For instance there had been four locations for one of the canals offtakes, three for another and so on.

Besides, the above mentioned reasons the system of irrigation and the water resources development are faced by the high sediment load carried by river Gash. In the last 40 years the sediment concentration increased from only 0.5% to 5-7%, i.e. 10 folds. In some irrigation basin the field level raised by 1.5 m.

A new design canalization system to cover an area of approximately 200,000 ha is adopted taking the nature of the Gash river and it's incorporated difficulties into account.

2. AIM OF THE STUDY

- to evaluate the present irrigation system.
- to establish new design approach in the delta scheme to overcome the irrigation deficiencies created by the expansion of the area and the sediment problem.
- to meet the crop water requirement through the integration of the available water resources.

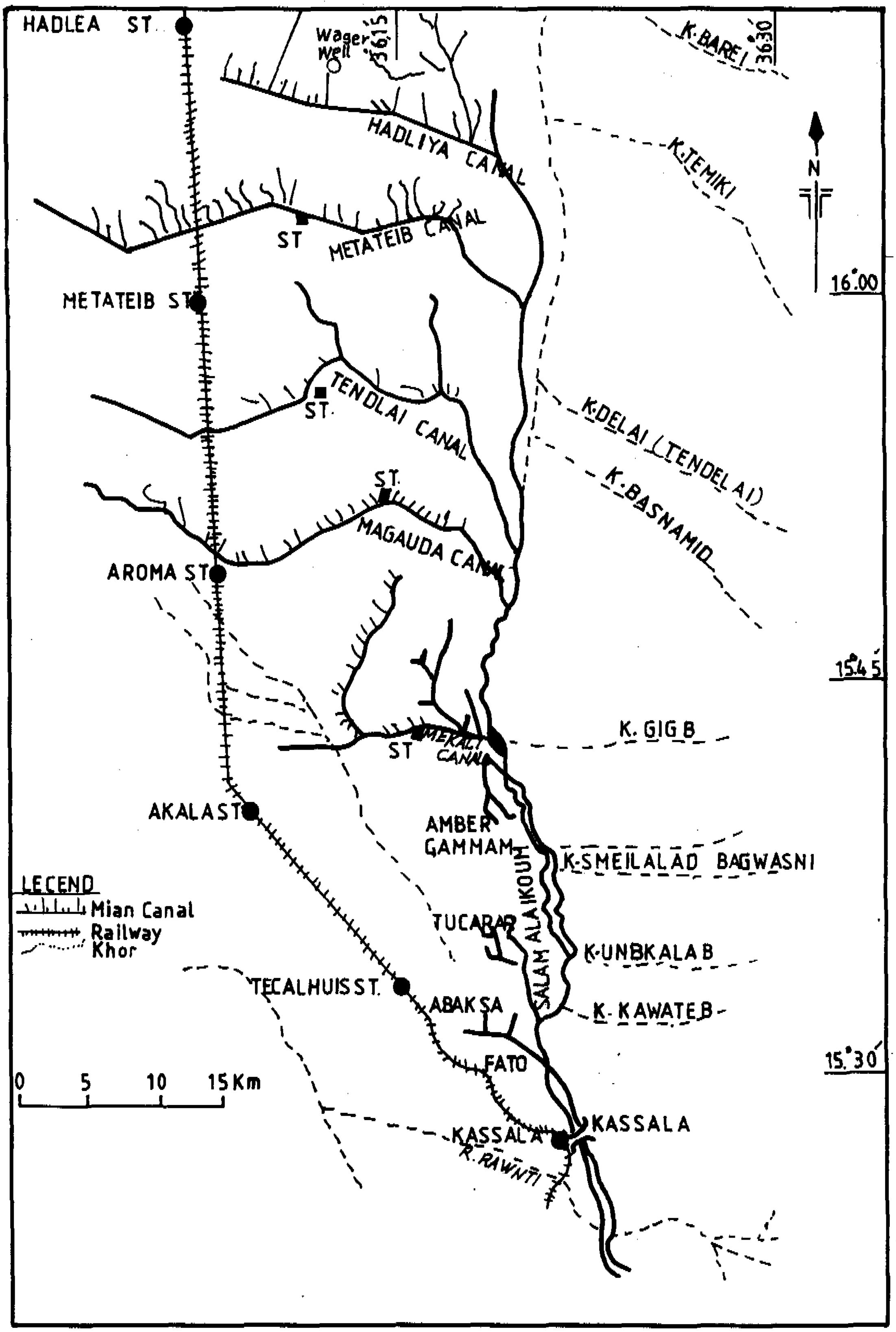


Figure 1. LAYOUT OF IRRIGATION SYSTEM IN GASH DELTA

3. WATER RESOURCES

Three types of water sources are found in delta Gash, although they are different in *availability and quantity.* Surface water is from river Gash. Rainfall and groundwater are in reasonable quantities.The rainfall occurs during the period which extends from June up to September and rarely in October. River Gash runs in the same period. Figure (2) shows the instability and variation of the Gash seasonal river flows together with rain quantities fall in the Delta area. The flow can start with only trickle move and reaches more than 1,000 m^3/s within one day.

Although Figure (2) indicates that the average flow of river Gash as 500 M m^3, recent discharge measurements showed that a factor of 2.5 should be applied to correct the records. For moderate and high river flows, the river depths increase due to scouring at the bed is considerably exceeded the rise in the water level e.g. if the level increases by 0.5 m the scouring depth increases by 2.0 m. Considering the last 10 years the average annual river Gash flow is estimated to one milliard m^3.

The rainfall is decreasing as from South to North, whilst in the Southern part of the Delta is 280 mm it is only 150 mm in the far part of the North.

In August 1993, for the first time,the Gash rehabilitation Corp. and the National Rural Water Corp., produced a detailed study of the groundwater in the Delta area. The study shows that a considerable amount of groundwater at different locations and various depths are available. Plenty of groundwater is available near the river Gash course and decreases as one goes away from it. However, in some locations inside the Delta the groundwater is salty and deep.

4. PRESENT IRRIGATION SYSTEM

The present irrigation system is composed of six main canals running more or less parallel to one another across the Delta from East to West, Figure (1). Bed slope generally varies from 1:1000 to 1:2000. The heavy sediment load carried by the river creates a lot of irrigation problems. It always leads to completely closure of the canals.The control structures along the main canals which lead directly to field basin are usually operated by stop -logs. On the other hand the water distribution is even worse and sometime the basin (1000-1500 ha) is watered for more than 40 days continuously. The latter practice leads to heavy water losses through evaporation and deep percolation. The former reaches more than 400 mm while the latter sometimes reaches 6 m deep, Swan, 1956, unpublished M.Sc. thesis. The rotational use of land by the lottery system is another factor complicated the present water management.

5. SOIL-CROP-WATER RELATIONSHIP

Swan, 1956, FAO, 1970, and Euroconsult et al, 1988, agreed that the Delta soil is one

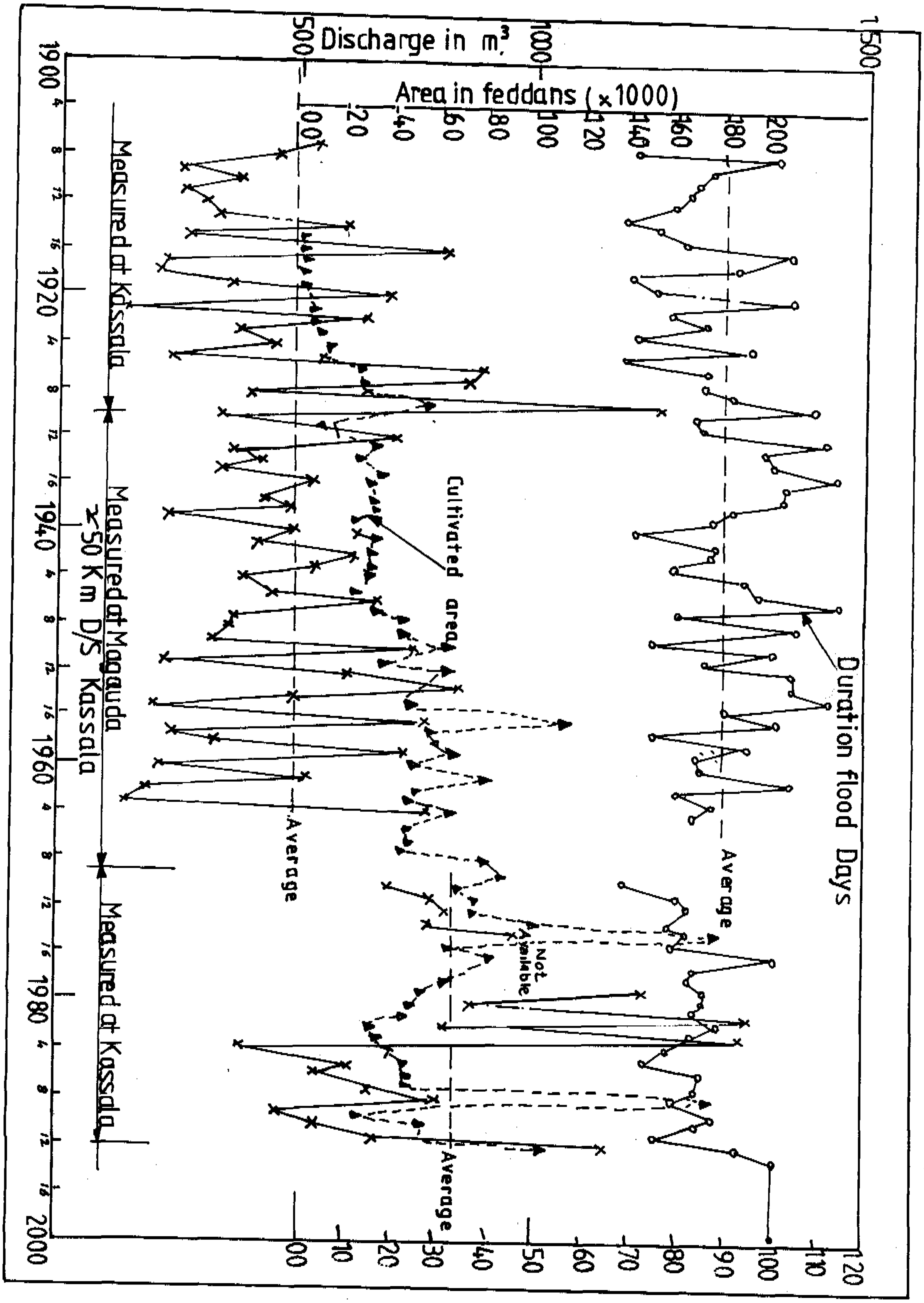


Figure 2. Gash flows, Duration flood days and cultivated areas.

Duration Days

of the best fertile soils in the world and composes of two main distinct types:-

- Silty clay loams, locally called Lebad.
- Heavy clay soil,locally called Badobe.

Until recently little was known about the soil moisture characteristics of the Gash Delta soils. The Euroconsult et al, 1988, reported that the water storage capacity in the Delta is 200 mm/100 cm soil column. In February 1994 Kees Hopmans of HVA (Dutch Firm) found that the water storage capacity (WC) in the Gash Delta soil is 50% higher than that quoted by Euroconsult et al i.e. 300-350 mm/ 100 cm.

If WC is considered as specific water-storage capacity multiplies by rooting depth in metre, the following roots depths are found from the data measured in the Gash Delta: Groundnut 150 cm; Guar 180 cm; Sunflower 180 cm; Karkadeh 100 cm; Okra 100 cm; Maize 100 cm; and Sorghum (Dura) 300 cm!

The above roots depths without limitation in soil moisture are very high when compared with other schemes in Sudan. This is definitely related to the soil characteristics and mainly the light soil with infiltration rate varies from 2.0 mm/ hr to 6 mm/hr. Such important information about the Gash soil needs more research work. The calculation of the crop water requirement and its comparison with that obtained from the soil moisture depletion is beyond the paper scope.

6. NEW PROPOSED IRRIGATION SYSTEM

The new irrigation system design is mainly based on the long experienced gained from the nature of the river Gash and the evaluation of the present irrigation system. The new design attempted to avoid as far as possible the drawbacks of the present system. It tries to answer the following questions:-

- To reduce the intake sediment load in the canalization system.
- To satisfy efficiently the crops water requirements through better water management.
- To serve the main goal by having permanent plot allocation for each farmer instead of the present lottery system.

The new design gives the opportunity to plant trees as wind breakers and shelter belts at the tail of the minor canal and double ABU XX, Figure (3). This also serves as protection from the creeping sand approaching the scheme. Such reserved forest will act as drainage areas for the scheme, since no water will be allowed to return to the river. The latter will be also a grazing area.

The new system allows for farmer participation in the water management, which was not possible in the old one.

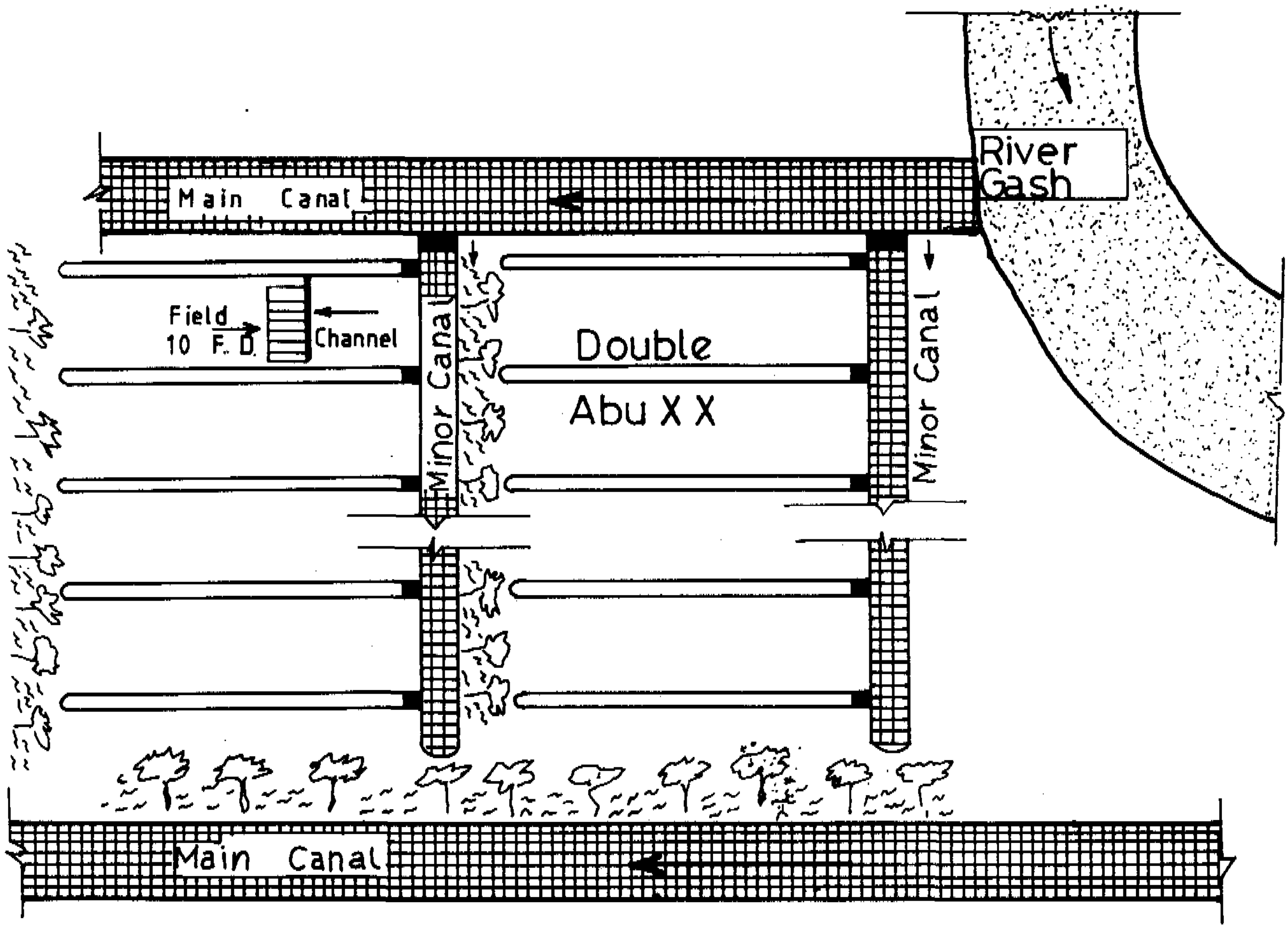


Figure 3. New Layout Design Of Gash Irrigation System

7. SEDIMENTATION

Sedimentation in most of the irrigation schemes creates a lot of problems, but here in the Gash Delta scheme determines the future of the whole system to be or not to be! In most of the time during the flood period the flow is mud rather than water with sediment. The sediment concentration is above 7%, while the whole river bed is set to motion. In some cases sediment deposits reached more than one metre depth over one night! Therefore sedimentation reduces the quantity of water that is diverted to the canals during the river Gash flood flow to less than 50% and the rest passed to the Gash Dia . In the past most of the irrigation canals were initially drawing clearer water from Balag areas (large flood basin). By time Balag areas were silted up and lost their sediment exclusion property. Many mitigation measures had been taken to reduce the quantity of sediment entering the system, however, the two main-ones were :-

- Siting the canal offtake to an appropriate river straight reach. That was done without knowing the river flow characteristics and angle of canal location.
- Using stop logs as weir in the entrance of the main canal. Here again a lot of problems are encountered which make such a method is less effective as silt exclusion.

New approach to reduce the sedimentation impact on the canalization system will be described later in this paper.

8. RIVER GASH MORPHOLOGY

From morphological features the Gash can be divided into three reaches. The first part represents 30 kms after it passes the Eritrean boarder with Sudan to Kassala town. The cross-section is wide 1500 m - 500 m, shallow 2.0 m to 3.9 m deep with few bends. The main course of the river is stable compared to the two other portions. The average bed slope is 1.3 m/km and its bed is mainly sands with average size of 0.1 mm.

In the second reach, downstream Kassala, 50 km, it becomes more serious with increase in number of bends and bank breaching at a number of points. The river width and depth varies between 700 m to 50 m and 2 m to 5 m respectively. The bed slope is averaging to 1.0 m/km.

In the third reach (40 km) the river course is more stable with almost straight, narrow and deep section. It width varies between 200-40 m with average width of 70 m. The depth varies between 3 m - 6.5 m. The bed slope is approximately 0.5m/km. This is attributed mainly to the river flow characteristics (water & sediment), the banks stability. While the upper reaches are composed of coarser materials with loose bank easy to be scoured, eroded and fall by sliding, the downstream reach has stable banks with fine materials (generally black clayey soil). To understand this behaviour of the Gash river course one should bear in mind that water always flows to the lowest point via a way that offers the least resistance. Due to differences in elevation and nature of the bed material, the watercourse will develop a kind of snaking pattern, the meandering nature.

Figure (4) shows the actual flow pattern in the Gash river near two of the canals offtakes. The water course will always try to find a balance between the cross-section and the longitude profile. For each point along the water course there exists a series of relationships between the discharge, the amount of transported sediment, the width and depth of the river bed, the roughness of the bed and banks, the grain size of the sediment, the current velocity and the river bed slope.

9. SITING AND DESIGN OF OFFTAKES

From his long experience in the Gash Delta the author of this paper believes that the proper siting and design of the canal offtakes with respect to the river course is one of the most difficult problems, especially when dealing with unstable river like the Gash.

Swan, 1956, who wrote a famous book about the behaviour of the river Gash and Spate irrigation system and its development, gave useful observations on the canal offtakes and problems encountered.

The Gash Rehabilitation Corp. and the Hydraulic Research Station, MOI, carried out an extensive study regarding the canal offtakes. The conclusion is to design and construct the main canals offtakes at a narrow and deep outer river bent. Figure (5) shows sketch of the proposed siting. The approaching reach U/S of the offtake and the D/S should be fully trained (preferred 300m U/S and 200m D/S). It was proved that such a location reduced the sediment quantity entering the canal and at the same time increases the flow diverted into the canal due to the different forces acting at the bend (Centrifugal forces), Figure (6). Therefore the canal offtake should be sited on the outside of a curve of fairly large radius (approximately 250 m) and should point slightly upstream when viewed from the canal, in such a manner that the main current in the river strikes the downstream part of the offtake structure from which it rebounds into canal. Hoping such setup to draw the maximum possible supplies even if the river is at low stages, but should never take in excess sediment. To furnish better conditions for the canal offtakes to work with satisfactory manner the diversion angle, i.e. the angle between the canal and the tangent to the outer bend at the offtake, is found ranging between 50^{0} to 54^{0}. The offtake should be located directly on the bent without any inlet channel. Such inlet channel proved to be a sediment trap basin.

Because of the varied river flow and the changing river geometry it is also recommended to make the total width of the offtake structure greater than the existing hydraulic conditions indicate.

10. SUMMARY AND CONCLUSIONS

As a conclusion for this paper some of the important points can summarized as follows:-

* Integrated water resources utilization in the Delta Gash is considered essential to increase the agricultural production.

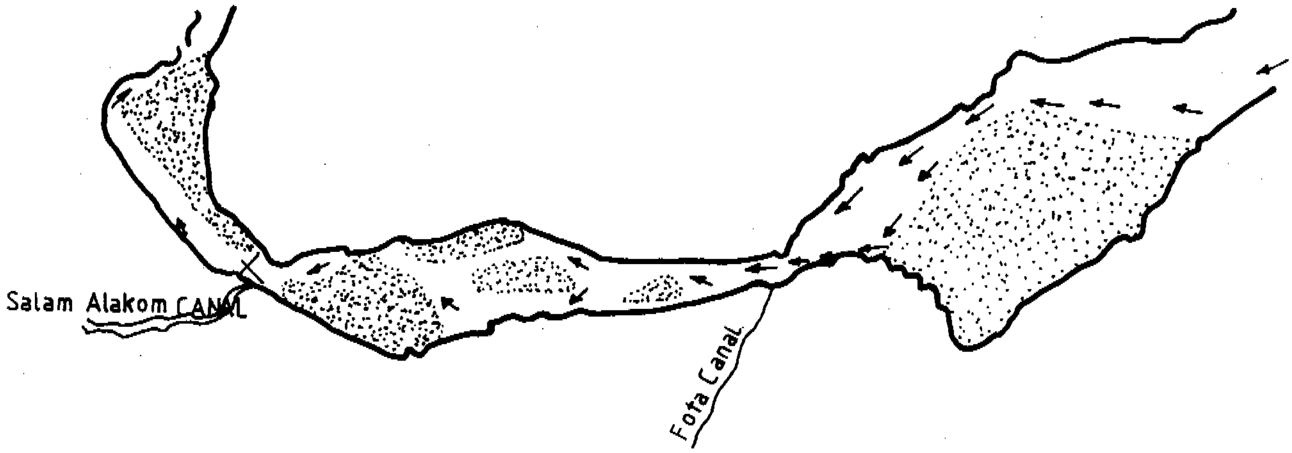


Figure 4. Flow pattern in Gash River with respect to the exiting canal offtakes

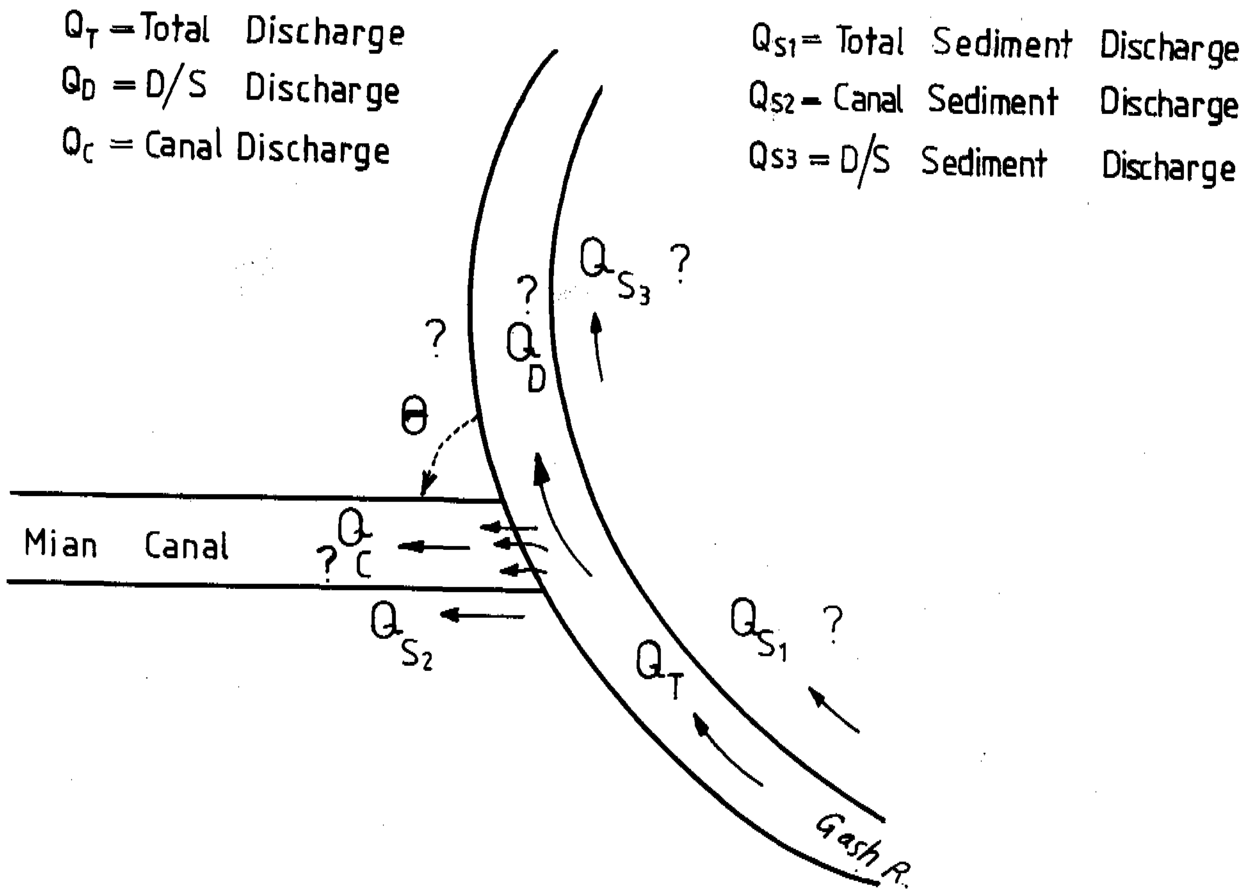
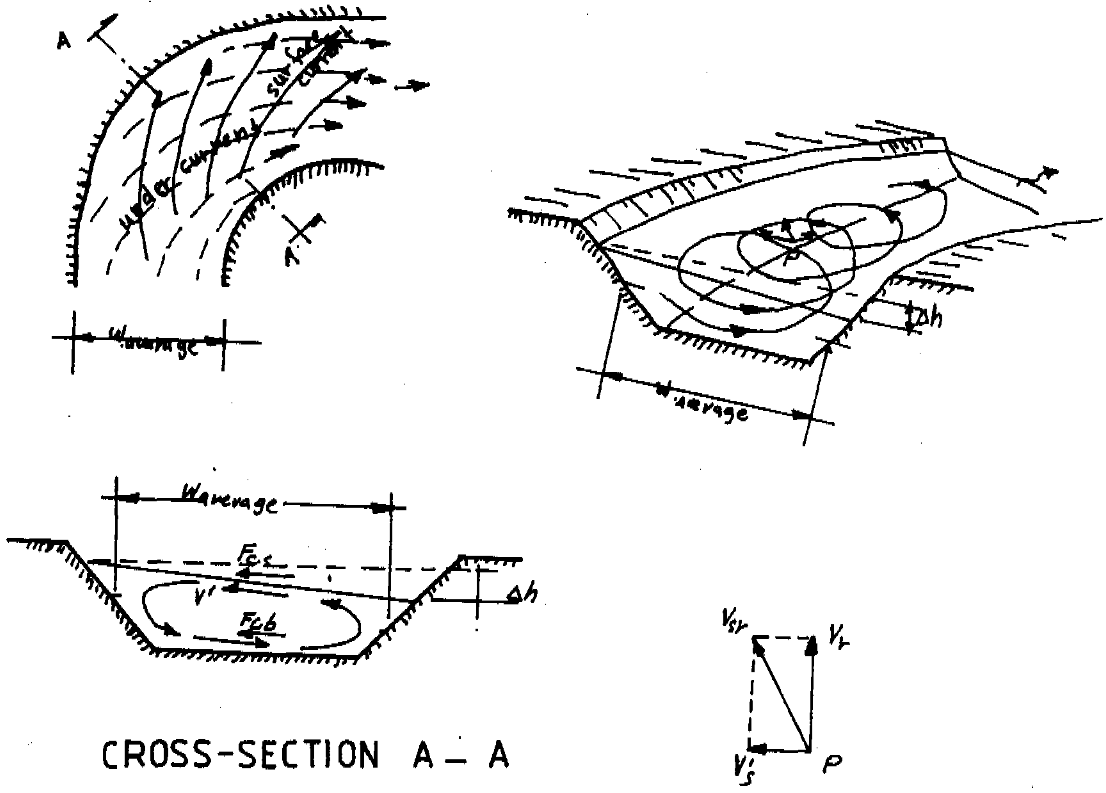


Figure 5. Location of Canal Offtake



$F_{c.s}$ = centrifugal force, near surface

$F_{c.b}$ = centrifugal force, above riverbed

$F_{c.s} > F_{c.b}$

Δh = difference in piezometric level

V' = flow, developed due to Δh

V'_s = surface flow in cross section A—A

V_r = flow in longitudinal section

V_{sr} = resultant surface flow

Figure 6. Flow Charachteristic in the riverbend.

* Gash river is a wide, steep, and unstable in most of its course. The high velocity (more than 4 m/s) and high sediment load (> 70,000 ppm) make it quite a challenge to design and operate irrigation system in a proper way inside the Delta.

* The new irrigation system, which is based on a considerable amount of data, is far better than the present irrigation system, especially in the water management and sedimentation aspects.

* Siting of the canal offtake at the river outer bend is found to be the most effective method to reduce (hydraulically) the sediment quantity entering the irrigation system and at the same time increases the water supplies diverted to the canals.

However, it is impossible to stop the fine sediment to pass to the irrigation system and hence to the fields. In this case sediment is recommended to be evenly distributed at the field level.

* The problem of sedimentation should always be under focus and more attention should be given to sediment monitoring and assessment.

* Farmers participation is very important in the water management. The new irrigation system allows them to do so!

* The high field water holding capacity of the Gash soil should be used to support two different crops (summer and winter). This can be assisted by the supplementary irrigation from the groundwater.

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LA GESTION DES BESOINS ET D'APPROVISIONNEMENTS EN EAU DANS LE BASSIN EUPHRATE-TIGRE

Ünal ÖZİŞ[1]

RÉSUMÉ

Le potentiel moyen annuel d'eau de l'Euprate est de l'ordre de 30-35, celui du Tigre de 50-55 km^3/a. Les réservoirs en Turquie vont créer au total un volume utile d'emmagasinage de 70 km^3 sur l'Euphrate et de 20 km^3 sur le Tigre; ceux dans les pays avals vont ajouter 15 km^3 sur l'Euphrate, notamment en Syrie, et 20 km^3 sur le Tigre, notamment en Iraq. Les pertes d'évaporation de ces réservoirs seront en Turquie aux environs de 4.5 km^3 dans l'Euphrate et 1.5 km^3 dans le Tigre; celles des pays avals seront de l'ordre de 2.5 km^3 dans l'Euprate et 4.5 km^3 dans le Tigre. A part de la production d'énérgie hydroélectrique et du controle effectif des crues, ainsi que certains approvisionnements en eau urbains et industriels, l'Euphrate va fournir l'eau pour irriguer 1.400.000 ha en Turquie et 800.000 ha en Syrie, le Tigre va la fournir pour irriguer aux environs de 600.000 ha en Turquie et 3.000.000 ha en Iraq, dont une part sera le long d'Euphrate. Les besoins d'eau d'irrigation nécessitent une gestion effective, éventuellement l'application des techniques qui consomment moins d'eau, en vue de l'allocation raisonable et équitable, citée ci-dessus, du potentiel d'eau, qui est limité à une certaine mesure. Pourtant, la part mésopotamienne de la crise d'eau du Moyen Orient parait comme un problème inopportunément exagéré, comparé aux autres conflits de la région.

ABSTRACT

The annual average water potential of Euphrates is about 30-35, that of Tigris 50-55 km^3/a. Reservoirs in Turkey will create active strorage volumes of 70 km^3 on Euphrates, 20 km^3 on Tigris; those in downstream countries will add about 15 km^3 on Euphrates mainly in Syria, 20 km^3 on Tigris mainly in Iraq. The evaporation losses from reservoirs in Turkey are about 4.5 km^3 in Euphrates and 1.5 km^3 in Tigris; from those in downstream countries are about 2.5 km^3 in Euprates and 4.5 km^3 in Tigris. Besides water power generation and effective flood control, as well as some allowances for urban and industrial water needs, the Euphrates will basically supply irrigation water to 1.400.000 ha in Turkey and 800.000 ha in Syria, Tigris will supply irrigation water to about 600.000 ha in Turkey and 3.000.000 ha in Iraq, part of it lying along Euprates. The demand for irrigation water should be managed, eventually techniques consuming less water should be adopted, within the context of the aforementioned reasonable and equitable allocation of the somewhat limited water potential. Anyhow, the Mesopotamian part of the Middle East water crisis appears to be an irrelevantly exagerated problem, compared to other conflicts in the region.

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1. INTRODUCTION

L'aménagement du bassin Euphrate-Tigre, formé par les deux cours d'eau qui se confluent près de Basra pour former le Chatt-el-Arab, intéresse principalement trois pays: la Turquie, la Syrie et l'Iraq.

Les travaux hydrauliques sur les cours d'eau principaux ainsi que sur leurs affluents, serviront:
(a) au contrôle des crues, surtout à l'aide des retenues créées par les barrages, ainsi qu'à la régularisation saisonnière et inter-annuelle des débits pour d'autres buts;
(b) à la production d'énergie hydroélectrique, soit par des usines installées aux pieds des barrages, soit par des usines de dérivation;
(c) à l'alimentation en eau des centres urbains et industriels, soit à partir des sources et de l'eau souterraine, soit par dérivation des eaux de surfaces retenues dans les réservoirs des barrages;
(d) à l'irrigation des terrains agricoles, grâce à la régularisation effectuée par les grands barrages;
(e) à la production de certaines espèces de poissons dans les lacs de barrages;
(f) à la navigation sur les grands lacs de barrages et sur certaines sections avales des cours d'eau.

L'étude des conditions hydrologiques, topographiques et géologiques montre que l'emplacement des grands barrages pour la régularisation des débits doit être effectué notamment en Turquie; de grands terrains agricoles qui peuvent être irrigués, existent dans tous les trois pays; certains centres urbains et industriels dans les trois pays doivent être alimentés de ce grand bassin.

Les grands réservoirs des barrages en Turquie servent aussi à la régularisation des débits pour les pays avals; mais les objections de Syrie et d'Iraq, en commençant par la construction de Keban et en continuant pendant la construction de Karakaya (Öziş et Özel 1989), sont intensifiées en vue de la construction du barrage Atatürk (Öziş, Basmacı, Harmancıoğlu 1990, 1992; Öziş et Harmancıoğlu 1994).

Il est évident que, si la Turquie utilise une partie de l'eau surtout pour l'irrigation, il y aura une certaine diminution du potentiel d'eau pour les pays avals, et une certaine diminution de la qualité d'eau dûe aux eaux de retour d'irrigation. D'autre part, si les grandes régularisations en Turquie n'existent pas, les pays avals ne pourront pas bénéficier d'une grande partie du potentiel d'eau. Cette dualité du problème de développement du bassin Euphrate-Tigre est aussi la clé de la porte entre coopération et conflit.

2. RELATIONS MULTINATIONALES

Le potentiel d'eau :
(a) limité dans le bassin d'Euphrate-Tigre,si on veut croître significamment l'irrigation

en Turquie, Syrie et surtout Iraq;
(b) plus limité dans le bassin du Nil, si on veut croître l'irrigation en Egypte, Sudan et Ethiopie;
(c) sévèrement limité dans le bassin de Jordan, en vue de la croissance de la population, soit naturellement, soit par immigration,
a servis comme sujets des scénarios de "crise" aboutissant aux "guerres d'eau" dans le Moyen-Orient, notamment dans les années '80 et '90 (Naff et Matson 1984; Starr et Stoll 1988; Anderson 1988; Starr 1991; Gruen 1991; Beschorner 1992; Beaumont 1992; University of Waterloo 1992; University of Illinois 1993; Kliot 1993b).

L'initiation du Projet de l'Anatolie Sud-Est (G.A.P.= Güneydoğu Anadolu Projesi) (D.S.I. 1980; Öziş 1982, 1983, 1986, 1991, 1992, 1993; Harmancıoğlu et Öziş 1983; Harmancıoğlu 1985; Kolars et Mitchell 1991) a entrainé une série de négociations bilatérales ou trilatérales entre la Turquie et la Syrie ainsi que l'Iraq, qui ne sont pas arrivées à un concensus, malgré l'approche positive de la Turquie pour la détermination raisonnable des besoins en eau de chaque pays et l'allocation du potentiel d'eau pour les couvrir (Tekeli 1990; Turan 1993a).

La seule exception est un accord conclus entre la Turquie et la Syrie en Juillet 1987, pour contribuer à la stabilisation politique et à la securité des frontières du pays, prévoyant la décharge d'un débit dépassant 500 m^3/s du bassin d'Euphrate à la frontière Turco-Syrienne (si pendant un mois ce débit devenàit inférieur à 500 m^3/s, la différence serait restituée le mois suivant), jusque le réservoir du barrage Atatürk serait plein et que les trois pays se consentiraient sur une allocation définitive des eaux de l'Euphrate (Protocol de Coopération Economique, Art. 6) (Gruen 1993).

Pendant la phase initiale de la mise en eau du réservoir Atatürk, les vidanges de fond devraient être clos pour un délai de près d'un mois. La Turquie a averti ses voisins de ce fait et, pour compenser en avance les eaux de cette période, a laissé couler aux environs de 830 m^3/s à partir du 23 Novembre 1989 jusqu'au jour de la mise en eau de la retenue, le 13 Janvier 1990. Malgré cet avertissement et cette compensation, plusieurs journaux du monde arabe et d'autres pays ont en fait d'usages sensationnels et antagonistes. Ces attitudes, avec des scénarios de "guerres d'eau", n'ont pas complètement cessées, même après l'ouverture des vidanges de fond en laissant écouler de nouveau 500 m^3/s vers les pays avals.

Peu après cet événement, la Syrie et l'Iraq ont apparemment conclu un accord sur l'allocation des eaux d'Euphrate provenant de la Turquie, selon lequel la Syrie utiliserait 42 %, l'Iraq 58 % de ces eaux (Beschorner 1992; Gruen 1993).

Les débits des cours d'eau, surtout dans le Proche-Orient, montrent grandes variations saisonnières et annuelles. Ainsi, il est absolument nécessaire de déterminer la quantité et la probabilité d'une allocation d'eau à l'aide des recherches opérationnelles, basées sur des modèles mathématiques pour la génération des données synthétiques et l'application des calculs de probabilité. Cette nature stochastique des débits devrait être considérée dans tous les accords internationaux sur l'allocation des eaux de cours d'eau qui écoulent à travers les frontières (Dudley 1993).

Il existe aussi certaines propositions pour le transport d'eau de la Turquie vers le Moyen-Orient. Malgré une disparité considérable entre les différentes régions du pays, la Turquie paraît avoir des ressources en eau supérieures à ces voisins, au point du potentiel d'eau par habitant du pays.

Une de ces propositions, nommée "adduction d'eau de la paix" (Duna 1988; Issar 1993), envisage le transport des eaux en excès des bassins Seyhan et surtout Ceyhan à l'aide de deux conduites forcées jusqu'au Golfe. Il est peu probable que Seyhan aurait un excès d'eau, si on tient compte des besoins dans le bassin-même et certaines dérivations aux bassins-clos de l'Anatolie centrale. Dans le cas de Ceyhan, une certaine part des eaux pourrait être considérée comme excessive, mais la dérivation devra avoir lieu définitivement après le barrage d'irrigation Cevdetiye, préférablement après la confluence du tributaire Kilgen, pour ne pas priver d'eau les systèmes d'irrigation et les usines hydroélectriques de ce bassin.

Une autre proposition envisage le transport maritime des eaux de Manavgat (Cran 1993) à différents ports de la Méditerranée-Est. La dérivation devra être effectuée à l'aval de la cascade et du lieu de la dérivation du système d'irrigation de la plaine de Manavgat.

Malgré les positions rivales de la Turquie, la Syrie et l'Irag à propos des eaux d'Euphrate, il existe même une proposition de transporter l'eau à partir de la retenue Atatürk, qui éviterait la nécessité d'adduction par refoulement et qui permetterait de produir aussi de l'énergie électrique (Wachtel 1993). Cette proposition oublie que la dérivation proposée de 1.1 milliard m^3/a pour la Syrie, le Jordan, l'Israel et la Palestine, privera les usines Atatürk, Birecik, Karkamış en Turquie de 600 millions de KWh/a, et ne prévoit aucune compensation matérielle pour cette perte continuelle.

L'eau est considérée dans le monde, d'une part comme une commodité de commerce (Zeitouni et al. 1993) qu'on vent et achète à un certain prix, et d'autre part comme un objet qu'on ne doit pas commercialiser et qu'on doit la fournir à celui qui en a besoin, selon les règles législatives d'Islam (Naff 1993).

De toute façon, les eaux des fleuves de la Turquie paraissent à continuer d'être la source principale pour couvrir les besoins du Moyen-Orient, avec le prix d'eau obtenue par désalinisation formant une limite économique à d'autres alternatives.

En outre, il existe aussi une tendance en vue de l'irrigation, que les pays avec des ressorurces en eau limitée et des terrains causant des problèmes de drainage, ne doivent pas nécessairement développer une irrigation intensive, mais qu'il serait beaucoup plus économique d'importer les produits agricoles, surtout s'ils ont des moyens, comme le pétrole, pour obtenir les devises à couvrir ces importations. Un exemple typique serait le cas de l'Iraq qui importerait les produits agricoles cultivés en Turquie dans le cadre du Projet de l'Anatolie Sud-Est, au lieu de tenter de développer excessivement l'irrigation.

3. BILAN GENERAL DES EAUX DU BASSIN EUPHRATE-TIGRE

On rencontre quatre opinions sur les eaux des bassins multinationaux dans le domaine du droit international: (a) la doctrine de la souveraineté absolue; (b) celle de l'integrité territorialle; (c) l'utilisation équitable et raisonnable sans causer de dommage appréciable; (d) la formulation de l' aménagement intégral et optimum du bassin sans tenir compte des frontières des pays (Caponera 1985; Cano 1989; U.N. 1991; Caponera 1992; Solanes 1992; Naff 1993; Kliot 1993, Gruen 1993).

La Turquie est accusée de se baser sur la doctrine de la souveraineté absolue à propos des eaux du bassin Euphrate-Tigre. Mais si on considère les travaux proposés dans le cadre du Projet de l'Anatolie Sud-Est surtout ceux des techniques d'irrigation moderne visant sur la conservation d'eau (Ünver et al. 1993), l' attitude de la Turquie est bien compatible avec l'approche d' utilisation équitable et raisonnable ainsi qu'avec celle de l' aménagement intégral du bassin (Tekeli 1990; Turan 1993a).

Le potentiel annuel moyen d'eau de l'Euphrate, provenant de la Turquie, de la Syrie et de l'Iraq, celui du Tigre provenant de la Turquie et de l'Iraq (y compris les apports des tributaries qui commencent en Iran, sauf ceux de Karoun), basé sur les observations hydrométriques et évaluations hydrologiques (Garbrecht 1968; D.S.I. 1980; Baran 1978; Shahin 1989; Kolars et Mitchell 1991; Wakil 1993), sont donnés dans les Tableaux 1 et 2, modifiés et élargis d'aprèsles les publications antérieures de l' auteur (Öziş 1982, 1983, 1991, 1992, 1993).

L'évaporation à partir des surfaces des retenues, les dérivations pour l'irrigation ainsi que les eaux de retour, la différence résiduelle des adductions d'eau pour couvrir les besoins domestiques et industriels, sont aussi montrées dans ces tableaux.

On en déduit qu' après l'aménagement complet du bassin Euphrate-Tigre dans les trois pays, les eaux qui vont couler par Chatt-el-Arab au Golfe seront, qualitativement, constituées presqu' entièrement des eaux de retour d'irrigation et, quantitativement, de l'ordre de 10 % du potentiel total de 80-90 milliards de m^3/a.

4. SOMMAIRE ET CONCLUSIONS

La Turquie est en train de développer ses ressources en eaux et en terrains agricoles dans le bassin Euphrate-Tigre, selon le plan d'aménagement intitulé le Projet de l'Anatolie Sud-Est (G.A.P.).

Ce projet prévoit une production totale d'énergie électrique de 37 TWh/a, avec 20 TWh/a dans la Basse-Euphrate, 8 TWh/a dans le Tigre-Ouest et Moyen, 9 TWh/a

Tableau 1 : Bilan d'eau général et approximatif du basin de l'Euphrate pour une année d'hydraulicité moyenne

		Volume d'eau (km^3/an)
Écoulement naturel à Karkamış		27.0 ~ 30.4
Écoulement naturel des affluants passant à Syrie	+	0.5 ~ 1.0
Évaporation des retenues de la Haute-Euphrate	-	0.7
Irrigations de la Haute-Euphrate (300.000 ha)	-	2.5
Eaux de retour de la Haute-Euphrate	+	0.3 ~ 0.8
Évaporation de la retenue Keban	-	1.0
Évaporation des retenues de la Basse-Euphrate	-	2.5
Irrigations de la Basse-Euphrate (1.000.000 ha)	-	10.7 ~ 13.0
Eaux de retour de la Basse-Euphrate (à l'Euphrate)	+	0.5 ~ 0.9
Eaux de retour de la Basse-Euphrate (aux affluents)	+	0.8 ~ 1.3
Pertes d'adductions d'eau domestiques et industriels	-	0.4
Écoulement du bassin d'Euphrate de la Turquie à la Syrie	+	~ 13
Écoulement naturel provenant de la Syrie	+	2.0 ~ 4.0
Évaporation des retenues en Syrie	-	2.0
Irrigations en Syrie (800.000 ha)	-	11.0 ~ 12.0
Eaux de retour en Syrie	+	1.5 ~ 2.5
Pertes d'adductions d'eau domestiques et industriels	-	0.3
Écoulement du bassin d'Euphrate de la Syrie à l'Iraq	+	~ 4
Écoulement naturel provenant d'Iraq	+	0 ~ 1.0
Évaporation des lacs en Iraq	-	0.5
Petites irrigations en Iraq	-	1.0
Écoulement d'Euphrate à Chatt-el-Arab et au Golfe	+	~ 3

Tableau 2 : Bilan d'eau général et apprroximatif du bassin du Tigre pour une année d'hydraulicité moyenne

		Volume d'eau (km^3/an)
Écoulement naturel à Cizre		16.0
Écoulement naturel de l'affluent Zap à la frontiére iraqienne	+	3.2
Écoulement des autres affluents à travers les frontières	+	1.8
Évaporation des retenues du Tigre-Ouest et Moyen	-	1.0
Irrigations du Tigre-Ouest et Moyen (650.000 ha)	-	5.4 ~ 6.5
Eaux de retour du Tigre-Ouest et Moyen (au Tigre)	+	0.5 ~ 0.7
Eaux de retour du Tigre-Ouest et Moyen (aux affluents)	+	0.2 ~ 0.3
Evaporation des retenues du Tigre-Est	-	0.5
Ecoulement du bassin de Tigre à la Syrie et l'Iraq	+	~ 14
Écoulement naturel provenant d'Iraq	+	26 ~ 35
Évaporation des retenues en Iraq	-	3.0 ~ 5.0
Irrigations en Iraq (y compris celles dans le bassin d'Euphrate) (3.000.000 ha)	-	40 ~ 45
Eaux de retour en Iraq	+	4 ~ 9
Pertes d'adductions d'eau domestiques et industriels	-	0.5
Écoulement de Tigre à Chatt-el-Arab au Golfe	+	~ 4

dans le Tigre-Est, à l'aide de 60 usines hydroélectriques, dont 18 sont situées dans la Basse-Euphrate, 12 dans le Tigre-Ouest et Moyen, 30 dans le Tigre-Est.

A part de l'alimentation en eau de certains centres urbains et industriels, le projet envisage aussi l'irrigation de 1.800.000 ha, dont 2/3 dans le bassin d'Euphrate, 1/3 dans celui du Tigre.

Un total de 90 grands barrages, dont 53 dans la Basse-Euphrate, 15 dans le Tigre-Ouest et Moyen, 22 dans le Tigre-Est, serviront à créer les retenues nécessaires pour l'irrigation et/ou la production hydroélectrique.

Le Projet de l'Anatolie Sud-Est comprends des unités de grandeurs universelles comme le barrage Atatürk en enrochement, avec 84 millions de m^3 de volume du remblai, 48 milliards de m^3 de volume du réservoir, 2400 MW de capacité installée et 9 TWh/a de production annuelle, fournissant l'eau pour irriguer 1,2 million hectares de terrains agricoles; les tunnels-jumeaux Şanlıurfa, ayant chacun une longueur de 26,4 km et un diamètre intérieur de 7,6 m; le barrage poids-voûte Karakaya de 173 m de hauteur, avec un volume de 2,6 millions de m^3 en béton, 1800 MW de capacité installée et près de 8 TWh/a de production annuelle.

Le barrage en enrochement Keban de 207 m de hauteur, situé dans la Haute-Euphrate juste à l'amont de Karakaya, avec une capacité installée de 1360 MW et une production de 6 TWh/a est le réservoir clé avec un volume utile de 25 milliards de m^3, en vue de la régularisation des débits de l' Euphrate.

Après la réalisation complète du Projet de l'Anatolie Sud-Est envisageant l'aménagement du bassin Euphrate-Tigre en Turquie, sous les conditions d'hydraulicité moyenne, aux environs de 45% dans l'Euphrate et de 65% dans le Tıgre des eaux provenant de la Turquie vont continuer à écouler vers les pays avals. La nature stochastique des débits doit etre pourtant considerée par l'intermédiaire des niveaux de probabilité dans le cadre d' allocations eventuelles des débits de chaque bassin.

La capacité utile des retenues à l'amont des barrages en Turquie est aux environs de 70 milliards de m^3 dans l'Euphrate, soit plus du double du volume moyen annuel d'écoulement, de 20 milliards de m^3 dans le Tigre, soit de l'ordre-même de ce volume. Il n'est pas possible, surtout sur l'Euphrate, de créer cette capacité dans les pays avals; de plus, les taux d'evaporation sont bien superiéurs à ceux en Turquie.

Ainsi, la régularisation par les retenues en Turquie servira aussi les pays avals, soit au contrôle effectif des crues, soit à l'augmentation des débits pendant les périodes sèches. De toute façon, l'evaporation à partir des réservoirs de la Turquie ne doit pas être seulement débitée à son compte, ni considérée comme une perte inutile du potentiel d'eau.

Compte tenu du potentiel limité d'eau et d'une subsistance pour couvrir les besoins en eau de la population rapidement croissante dans la région (Engelman et LeRoy 1993), avec une gestion efficace de l'irrigation, surtout par l'application des techniques qui consomment moins d'eau, la part mésopotamienne de la crise d'eau du Moyen-Orient paraît comme un problème inopportunément exagéré, comparé aux autres conflits de la région.

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WATER DEMAND MANAGEMENT IN CANADA - PAST, PRESENT AND FUTURE

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ABSTRACT

Water demand management and conservation have recently emerged as valuable additions to Canadian water resources management, which has traditionally concentrated on the development and/or manipulation of the country's large water supplies. Rising development costs, capital shortages, government fiscal restraint, diminishing sources of water supply, polluted water, and a growing concern for the environment have all contributed to the realization that water use is actually a demand that needs to be controlled.

This paper shows that water demand management holds the key to dealing with many Canadian water management problems, drawing upon several recently-completed pieces of research. The theme of water resource economics is examined in the context of municipal infrastructure viability. This discussion is broadened with reference to the quantification of industrial water demand functions, based on primary data collection. The final "economic" issue explored focuses on industrial water pollution as a demand management problem. The structural, operational and socio-political dimensions of demand management are also dealt with using Canadian research and examples. Finally, the paper examines recent Canadian efforts to model the potential effects of water demand management on water availability, through the use of Environment Canada's Water Use Analysis Model.

The principal conclusion of the paper is that significant progress in solving many Canadian water management problems will not be possible without substantial emphasis on the tools and techniques of water demand management. This conclusion is broadly applicable to other countries.

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RÉSUMÉ

La gestion de la demande d'eau et la conservation de l'eau sont de précieux éléments qui sont venus s'ajouter depuis peu à la gestion des ressources en eau au Canada, laquelle a consisté, par le passé, à mettre en valeur et à manipuler les vastes réserves d'eau du pays. L'augmentation des coûts d'exploitation, la pénurie des capitaux, la compression des dépenses gouvernementales, la diminution des sources d'approvisionnement en eau, la pollution de l'eau et l'augmentation de l'attention prêtée à l'environnement ont tous contribué à faire comprendre que l'utilisation de l'eau est en fait une demande que l'on se doit de contrôler.

Dans la présente communication, on montre que la gestion de la demande d'eau s'avère la solution à de nombreux problèmes de gestion des eaux auxquels est confronté le Canada, tout en s'appuyant, pour ce faire, sur plusieurs recherches qui viennent d'être complétées. On examine le thème de l'économie des ressources en eau dans le contexte de la viabilité des infrastructures municipales. Cet examen est élargi en ce qui concerne la quantification des fonctions de la demande d'eau industrielle, qui est basée sur la collecte de données primaires. La dernière question «économique» qui est étudiée sous tous ses aspects est celle de la pollution industrielle de l'eau en tant que problème de gestion de la demande. Les dimensions socio-politiques de cette gestion tout comme celles liées aux structures et aux modes d'utilisation sont aussi abordées et étayées d'exemples et de résultats de travaux de recherche canadiens. On examine enfin les efforts déployés récemment par le Canada pour déterminer, grâce au modèle d'analyse de l'utilisation de l'eau d'Environnement Canada, les effets possibles de la gestion de la demande d'eau sur la disponibilité de cette ressource.

La principale conclusion qui ressort de cette communication est qu'il sera impossible de beaucoup progresser dans la résolution des problèmes de gestion de la demande d'eau au Canada sans faire une large part aux outils et techniques mis au point à cette fin. Cette conclusion s'applique dans les grandes lignes à d'autres pays.

1. INTRODUCTION

Water demand management comprises a fairly new approach to managing water resources. While it is certain the water managers have, throughout history, carried out their duties to meet a variety of "demand" on the resource, they have not often tried to influence and channel those demands to satisfy economic or environmental objectives. In Canada, for example, a national perception that water occurs in plentiful supply has conditioned most water management programs to constructing new supply systems, building often-massive structural works - in other words to manipulating the supply of the resource to meet so-called human "needs". This "supply management" approach views water use as a requirement to be satisfied, not as a demand that can be influenced by policies or alternative actions.

In contrast, the root of the water demand management "paradigm" is the supposition that each use - municipal, industrial, agricultural - comprises an alterable demand that can be influenced in a variety of ways. Formally stated, water demand management consists of any socially beneficial measure which reduces or reschedules peak or average withdrawals from surface or groundwater while maintaining or mitigating the extent to which return flows are degraded. This definition emphasizes that (1) it is the average or peak water use that we should be concerned about - in any location, not just those in semi arid or arid areas, (2) measures taken must have a social benefit - for example a positive benefit:cost ratio, and (3) measures undertaken should maintain or improve water quality.

This paper analyzes the use of demand management in Canada, a nation where overall water supplies are abundant by most measures of water availability. However, even in Canada, regional variability in supply, a distribution of water tending away from the settled parts of the country, deteriorating water quality in many areas and vastly increasing infrastructure costs have all created a fertile environment for the adoption of water demand management policies and programs. The first section of the paper is conceptual in nature, discussing the need for demand management in the context of the need to integrate environmental and economic factors into the fabric of development - as the United Nations Commission on Environment and Development (UNCED) (1987) put it - to foster sustainable development. The concern here is two-fold. First, for practising analysts, it is important to examine the ways in which this integration can proceed. Second, it is equally important to demonstrate how demand management is vital to this integration. The second section of the paper explores a typology of demand management actions, using primarily Canadian examples. Finally, these examples are drawn together to provide an indication of what future actions are required to establish demand management as (1) an operational focus for water managers, and (2) an essential component of any strategy to move toward sustainable development.

2. WATER DEMAND MANAGEMENT AND SUSTAINABLE DEVELOPMENT

This paper is not about sustainable development per se, and the complexities involved in defining it are left to others. Instead, the UNCED definition is accepted at face value - namely that sustainable development refers to the growth and prosperity of today's generation without impairing the ability of future generations to do likewise (UNCED, 1987). As the Commission pointed out, the sustainable development concept has, at its heart, the idea that decision-making on the economy must incorporate (i.e. integrate) environmental conditions and vice versa.

At the heart of most economies, and certainly Canada's, lies the"market system", which governs the way in which the exchange of goods and services occurs. Because basic exchange systems are unlikely to alter radically in the future, it is productive to consider how economic principles can apply to the environment[2]. The following discussion uses Canadian examples to illustrate some typical water management issues: excessive water use, infrastructure financing and water pollution control approaches.

2.1 Excessive Water Demands

As noted above, the predominant approach to regional water shortages has traditionally been the erection of facilities to increase supply - pipelines, dams, diversions and the like. Under this approach, water is conceptualized as a requirement to be met, not a demand to be influenced by policy. In Canada, and throughout the world, often great efforts are made to manipulate available supplies spatially and temporally to where the water is "required". At least two major problems arise. In the process of developing new water supplies, ecosystems and wildlife habitat may be altered or destroyed, possibly leading to decreases in biodiversity. Second, new sites required to replace or supplement old ones (economists call these "marginal" opportunities) will probably be more expensive than earlier-developed sites. These "increasing marginal costs" may be very expensive in social terms. For example, in a Canadian federal institution, the finding that the marginal cost of a new water supply and treatment plant was $5 per cubic metre led to the search for ways to conserve existing supplies (Tate and Macmillan, 1992).

The root of this excessive demand problem is a very simple one to diagnose using economic concepts. Canadians pay water prices which are among the cheapest in the world. For example, the residential price of water and wastewater services in Canadian municipalities averages just over $0.60 per cubic meter (Tate and Lacelle, 1992) - literally cheaper than dirt! Substantial evidence exists (Federation of Canadian Municipalities, [FCM]1985) that these prices fail to cover the full costs of water system operation. More will be said about this later in the paper. A second example relates to the public subsidies given for irrigation water supplies - up to 90% of total cost in some areas of the country. At this price, there is little wonder that irrigation water demands are large. It is well to remember that the <u>end</u> of irrigation is food production; irrigation is only the <u>means</u>. If food can be obtained more cheaply, in social terms, by cheaper production methods, and there is little doubt that in Canada it can be, the use of irrigation may introduce large inefficiencies to the production process. It also results in excessive water use and consumption in one of Canada's driest regions.

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[2]This approach contrasts sharply with an alternative approach to environmental economics, which attempts to influence economic decision making with physical constraints, such as the laws of conservation of energy (see, for example, Daly and Cobb, 1990).

In terms of sustainability, therefore, excessive demands present substantial problems. In economic terms, excessive water demands result is wasted (scarce) capital resources, and thereby introduce substantial inefficiencies into the economy. Ridding the economy of these inefficiencies would free up resources for more valued uses. Environmentally, excessive demands lead to construction of public works, and the potential destruction of ecosystems, which have substantial sustainability benefits. The conclusion is that water demands higher than justified economically work against the principle of sustainable development.

2.2 Infrastructure Financing

Major problems of infrastructure adequacy and financing can be traced to the same problems which cause excessive water use. Low water prices, or the absence of water prices, create the illusion that water servicing is very cheap. The last section showed that there exists a direct link between low water prices and high water use. But low prices actually form one part of a two-edged sword. The other part is inadequate revenue for operating, maintaining and, where necessary upgrading and expanding municipal infrastructure. In Canada, this revenue inadequacy has reached major proportions. In 1985, the Federation of Canadian Municipalities (FCM, 1985) estimated that some $7.5 billion were required to renew municipal water infrastructure. Internal Environment Canada estimates place this figure close to $10 billion currently, since no major country-wide efforts to solve the problem have been made. The FCM study found that about 70% of water supply costs, but less than 50% of waste treatment costs were being recovered from user fees - water rates as they are referred to in Canada. The implication is that revenue inadequacies attributable to low water prices are directly responsible for infrastructure deterioration. Further, internal Environment Canada studies show that, to solve the problem would require some modification and increase in water rates, but that these revisions would not cause excessive hardship to any class in the Canadian society (Beaulieu et al., 1993).

The primary concern here in sustainability terms is public health. If water systems are allowed to deteriorate over time, the result is increasingly inadequate water supplies and potential break-downs in waste treatment systems, both posing public health problems. Thus, again, direct link is clear between inadequate practices in the water industry and a central problem of sustainable development - the assurance of safe drinking water and adequate waste treatment and disposal. Also, as will be documented below, it is an area where water demand management can play an important role in solving the problem.

2.3 The Industrial Pollution Control Problem

One of the more serious current water management problems, not only in Canada but in many other nations has been the incorrect diagnosis of the cause of pollution problems. This issue cuts straight to the heart of the sustainable development debate.

Environmental resources form <u>essential</u> inputs to industrial production. Industry could not operate in the absence of these resources. Economists employ a conceptually straightforward model of the manner in which basic resources are transformed into goods and services:

$$Y = f(N, L, C)$$

where:

$Y =$ the quantity of any good or service produced;
$N =$ the quantity of natural resources used during production;
$L =$ the quantity of labour used during production;
$C =$ the quantity of capital used during production.

The natural resource factor N includes the use of environmental resources. Goods can be produced in a variety of ways by combining the factors in different proportions, even for essentially the same product. The fundamental factor governing relative inputs is the cost, or price, of those inputs. Accordingly, since they are profits maximizers, industries will attempt to minimize their production costs by seeking the lowest cost combination of inputs which will satisfy their output requirements. When an input is cheap, more of it tends to be used. When it is very cheap, it is used as required, with no thought for conservation.

The elementary economic model described here provides an important insight into the water and air pollution problems that face the world currently. Because environmental inputs are very cheap, and often free, they become used to excess, in a manner outlined earlier for water intake. But the same principal applies on the discharge side of the water use cycle. There exist a large range of options for ridding an industrial operation of its wastes, and many of these options involve large outlays of money. If, at the same time, the option exists of continuing to pour untreated or semi-treated wastes into receiving waters or the atmosphere free of charge, there is little doubt that industries will choose the "free" alternative. This will remain so even as well-intentioned public agencies continue to negotiate schedules for compliance with legally set regulations. In Canada, at least, few prosecutions for polluting behaviour are undertaken by either federal or provincial agencies, and it is suspected that the same is true in other countries.

The fear is that tough stands on pollution control will lead to industrial relocation. Two factors suggest that this argument is overstated. First, basic locational analysis suggests that industries face a variety of factors in choosing a location, and that environmental factors may be relatively minor. Markets, labour availability and raw material stock seem to be much more important. Second, if most countries moved together in approaching pollution problems, the incentives for relocation, if they do exist, will be neutralized.

A supplementary observation is that water managers, in spite of their strong training in the applied sciences, have very little understanding of the forces that underlie technological change. Technology here refers to the "social pool of industrial arts" (Schmookler, 1966). These industrial arts change in response to a largely economic set of conditions (Solow, 1957), not in response to random events occurring in the sciences or by phenomena such as "luck" or "serendipity". Central to the conditions for technological change is the ability to value resources commensurate to their contributions to production. Without such valuations, change happens only slowly over time. Technologies for water conservation, which change very slowly, are prime examples of what happens when resources are substantially undervalued.

Not unexpectedly, the same model that describes the production process suggests the only viable solution to the environmental pollution problem. Since inputs are combined in the production function in proportion to their price, the solution to the overuse of the waste-absorbing characteristics of the environment is to price the use of environmental resources appropriately. This means, for example, that the large variety of economic instruments for pollution control should be exploited in a sustained effort to overcome environmental degradation. Examples would include realistic pricing for water services, input charges for materials that contribute to the generation of toxic substances (e.g. chlorine), effluent discharge fees, tradeable emission permits, and other types of instruments. In a recent study, it was shown that the use of effluent discharge fees to complement baseline effluent regulations in the Canadian pulp and paper industry offers a cheaper, and technologically superior means of achieving social aims than the use of effluent regulations alone (Harris and Tate, 1994).

In other words, if sustainable development, economy-environment integration, and like terms are to mean anything practical, decision-makers at all levels must examine the full potential for using economic means to influence environmental decision-making. Nowhere is this more important than in the realm of pollution control. Indeed, without relying on economic incentive mechanisms, there is very little hope for preventing further environmental deterioration in the future.

3. A TYPOLOGY OF WATER DEMAND MANAGEMENT ACTIONS

The examples outlined above show that water demand management has an important role to play in solving current and future water resource problems. The new paradigms being adopted in environmental management, such as sustainable development, will be virtually impossible to implement without the demand-oriented approaches outlined in this paper. This section presents a simple classification scheme for water demand management actions, illustrating each type of action with Canadian examples.

Water demand management tools and techniques break down into three basic categories: economic, structural and operational, and socio-political. **Economic techniques** rely upon a wide range of monetary incentives (e.g. rebates, tax credits) and disincentives (e.g. higher prices, penalties, fines) to relay to users accurate information about the value of water. One of the main strategies of Canada's Federal Water Policy (Environment Canada, 1987) is based upon this type of demand management actions, in promoting the concept of realistic water pricing as a direct means of controlling water demand and generating revenues to cover costs. The discussions of municipal infrastructure financing and industrial pollution control, outlined earlier in this paper, rely heavily on the concept of using demand-based economic instruments for environmental improvement.

Structural techniques focus on altering existing equipment or infrastructure to achieve better control over water demand. Examples of structural measures include metering, flow control and recycling. They include a wide range of operational techniques what modify existing water use procedures to control demand patterns more effectively. They include leakage detection and repair as well as the implementation of water use restrictions during periods of water shortages. In the municipal water use field, metering of water supplies is a particularly important structural measure because it is the necessary first step in moving toward effective pricing arrangements. Without metering, any attempt at demand-based pricing and demand management will be futile. There is substantial evidence that metering, combined with demand-based pricing can lower water demand by over 30% from pre-metered levels (Figure 1).

Socio-political techniques in a water demand management context refer to policy and related measures which can be taken by public agencies to encourage water conservation. They include public awareness programs, laws such as building code and appliance modifications and government economic policies. These are designed to obtain cooperation from the public in moving toward improved water management practices. Thus one of the most important techniques in this field is effective public education.

4. A DEMAND MANAGEMENT ORIENTED VIEW OF THE FUTURE

As explained in this paper, many of today's water management problems have their roots in the way water is viewed in economic terms - as a requirement to be met, not as a demand that is subject to modifications. It is certain that the economic problems in managing water demands are not the only ones of concern, but they are much more important than is generally recognized. This is the reason why water demand management is so important as the world enters a new millennium. Demand management actions are required in virtually all economic sectors.

In the municipal sector, water users must begin to bear the full cost of service provision. In developed countries like Canada, there exist no good reasons why this principle cannot be applied immediately. In the case of smaller communities that cannot benefit from economies of scale, it would be better, where subsidies are contemplated, to subsidize consumers directly, thereby allowing them to demand the level of required service, than to subsidize municipalities to build supply systems. In this sector, it is particularly important that water supply and waste treatment be viewed as an integrated entity, not as separate services. Other major demand management techniques required include universal water metering, water conservation and efficiency programs, and public education programs.

In the industrial sector, it is essential that the prices paid for water use (both intake and discharge) reflect the value of the resource in providing these services. For publicly-supplied firms, this means that realistic water rates should be paid for industrial water, reflecting accurately the volume-related implications of supplying water to industry. For self-supplied firms, realistic prices imply that economic rent principles should be applied to determine the appropriate level of charge. On the discharge side, instruments such as extra strength sewer surcharges (for discharges to public sewers), effluent discharge fees, marketable effluent permits and the like should be exploited to the full extent possible.

Area	Impact and Special Details	Source
Western U.S.	- unmetered areas have over 50% higher water use than metered ones on average; over 100% for maximum day and maximum hour.	Linaweaver, Geyer and Wolff (1967)
Etobicoke, Ontario	- unmetered ares have 45% higher water use than metered areas of comparable assessment.	Grima (1972, p. 165)
St. Catherines, Ontario	- an 11% drop immediately following metering but rebounds because prices kept low. Two years later, water usage higher than before metering.	Pitblado (1967, p. 46)
Boulder, Colorado	- 34-37% drop on water use following meter installation	Hanke and Flack (1968)
Alberta	- 10-25% drop in water use following meter installation	Associated Engineering Services Ltd. (1980)
Peterborough Ontario	- 10% reduction in water use predicted following meter installation.	Peterborough Water Department (1984)
Central Valley, California	- household water use reduced up to 55% following meter installation; usage averaged 30% less in metered than in unmetered cities.	Minton, Murdock and Williams (1979)
Calgary, Alberta	- unmetered water use 46% greater than that in metered residences.	Mitchell (1984)
Calgary, Alberta	- unmetered water use 65% greater than that in metered residences.	Kellow (1970)
Dallas, Texas	- experienced a drop in water demand of 43% following meter installation.	Shipman (1978)
Gothenberg, Sweden	- per capita use in unmetered apartments 50% higher than in a single family, metered residences	Shipman (1978)
York County, Pennsylvania	- substantial increases in industrial waste treatment charges led to reductions in water use in the range.	Sharpe (1980)

Figure 1. The Effects of Metering on Municipal Water Use

For other sectors, there is an equally comprehensive set of demand management options available. It is well past time when public agencies, and the public itself, recognize that demand management offers the key to the very success of water management in the future.

5. MODELLING, DATA AND RESEARCH ON WATER DEMAND MANAGEMENT

Environment Canada has built up a program of water use studies, through a low-key persistent effort, over the last two decades. The program consisted primarily of data collection/analysis and research. In recent years, water demand management has been receiving special attention in these activities. The results of this continuing program provide a necessary input to future water demand management policy development.

Systematic water use data collection began in the 1970's. These data have significantly advanced our knowledge base and research opportunities. Water use surveys for the municipal sector (Tate and Lacelle, 1978, 1987) and for the industrial sector (Tate, 1977, 1983; Tate and Scharf, 1985) now collect economic and other data relevant to demand management in support of research and modelling efforts. Surveys and studies on municipal water pricing in Canada (Tate, 1989a; Tate and Lacelle, 1992) have also been carried out, which were used to analyze the price responsiveness of municipal water demands. Analysis of the industrial water use data (Tate, 1989b) revealed important facts about the current water use practices and identified measures which can have substantial impacts on reducing the amount of water used by industries. One observation from the data is that water is provided to industries practically free of charge. Water costs to industry in Canada are only 0.2% of the total annual input costs to industry. Renzetti (1986, 1987) was able to quantify the relationship between industrial water prices (cost) and the level of water use. Regression analysis of the industrial water use data showed statistically-significant impact on the level of water use, both nationally and provincially. The main policy implication is that water price could be used as policy instrument for controlling the demand for water by industries. The other relevant observation from the data is the potential for significant reduction in water use be means of recirculation. It is important to note that water recirculation is closely linked with the establishment of water quality standards. In general, the higher the costs of water supply and the more stringent water effluent standards or charges, the greater will be the propensity for water recirculation to occur.

In 1981, Environment Canada was commissioned to carry out a study to identify "Water Supply Constraints to Energy Development" in the energy resources rich regions of Canada. The study evolved into a multi-year research project and provided the opportunity to consolidate Environment Canada's data and methodological advances into a state-of-the-art computer model. The end result of the work is the Water Use Analysis Model (WUAM). WUAM is primarily a water use forecasting (projection) model which translates social, economic and policy variables (scenarios) into demands on the water resources. It also simulates water supplies and carries out water balance calculations. Water uses include urban-municipal, rural, industrial, irrigation, livestock, power generation and other sectors. All categories of water use can be broken down, when necessary, to provide a fairly fine level of sectoral details.

Unlike most existing models, WUAM places special emphasis on water demand modelling and allows the user to incorporate most of the variables influencing water demands in the water use projections. The model is explained in detail by Kassem (1992) and is represented conceptually in Figure 2. In the water use projections, WUAM employs 'activity level' forecasts (e.g. population or economic output), and coefficients of water use per unit of the activity level. The exception is the irrigation sector, which is modelled in some detail. The impacts of water demand management measures on water resources can be simulated in WUAM through the alteration of the water use coefficients, or, alternatively, WUAM can explicitly simulate the impacts of several water demand management measures such as water pricing, structural and operational changes, etc.

6. CONCLUSIONS

The concept of sustainable development offers a powerful new paradigm for resource management throughout the world. It is interesting to reflect on the history of economic development in many areas, and to realize that, in the sense that resources have not been depleted in most areas, development has been sustainable. The crucial problem, however, is that the environmental resources have fared poorly in the rush to develop new products and new processes. This is true not only in the industrialized world, but in less developed counties as well.

The world is faced with a real paradox which must be solved on the way to a sustainable future. The paradox is that the very mechanisms that have generated development have had harmful, and in some cases catastrophic, effects on environmental resources. Resolution rests in part in learning how to use the mechanisms and concepts of economic development to protect and enhance environmental resources, in the present case water resources. It is in this effort that water demand management offers great potential. For example, the set of economic techniques which form the backbone of demand management are of central importance in adapting established development techniques to water management. As shown earlier in this paper, these techniques can play a crucial role in dealing with issues on both the intake (e.g. lowering water demands) and the output (e.g. diminishing pollution) sides of the water use cycle.

From experiences in Canada, where demand management is still in its infancy, it is possible to say that problems of water shortage and pollution still exist to quite a large extent. It is also a fact that water demand management techniques have not been relied upon extensively in the past to deal with water resource issues. It appears that the solutions to many of these problems are at an impasse without the potential of demand management being recognized. This paper has dealt with many demand management approaches that can be used to meet and solve current water problems, and to enhance the sustainability of the resource. It seems fairly certain that, without the incorporation of demand management principles into the way current and future problems are approached, a sustainable future will not be possible. Therefore, the adoption of water demand management is critical for the sustainable development of water resources.

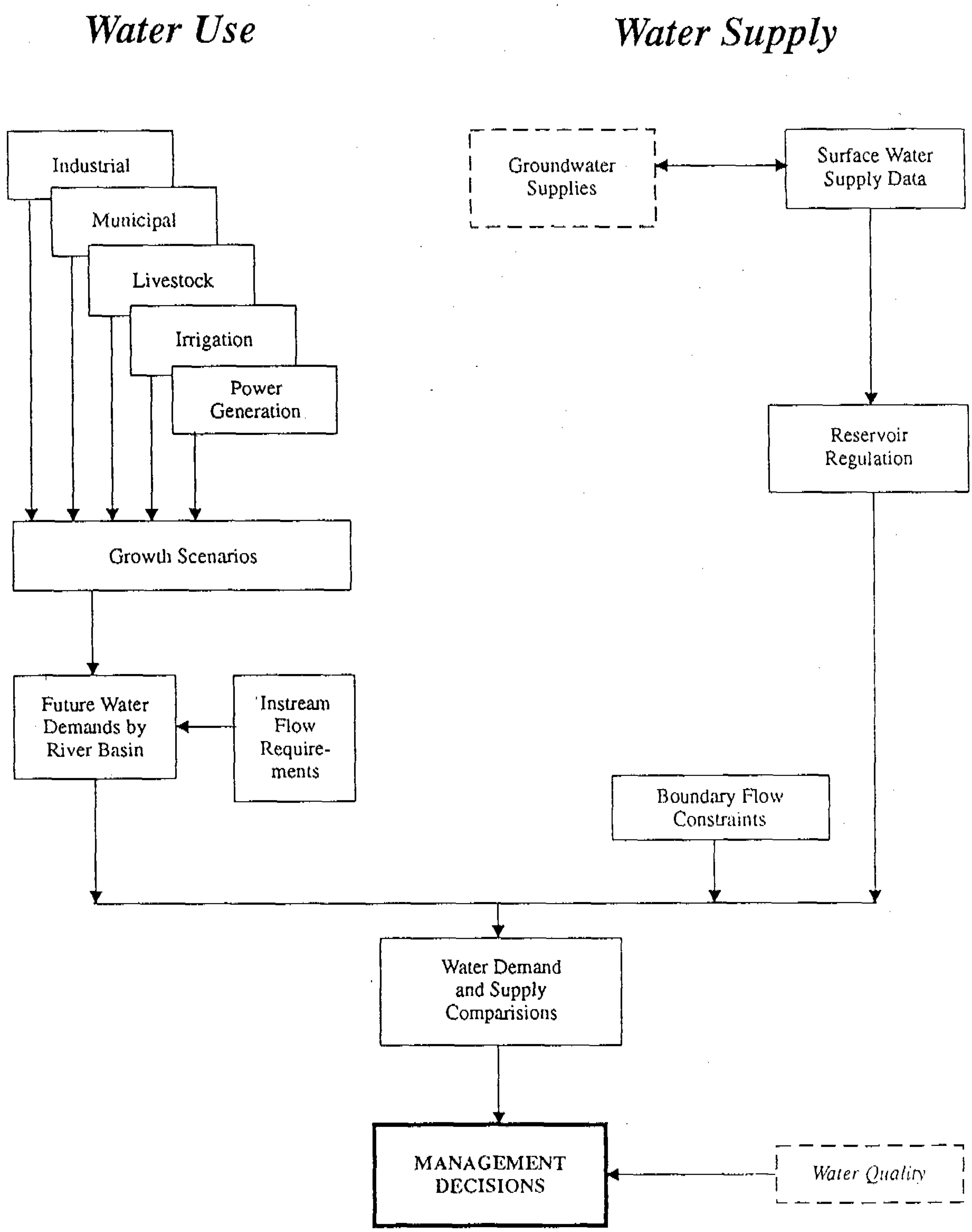

Figure 2. Conceptual Overview of the Water Use Analysis Model (WUAM)

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WATER RESOURCE MANAGEMENT IN THE MEDITERRANEAN BASIN

A. Hamdy *, M. Abu Zeid and C. Lacirignola*****

ABSTRACT

In the face of population growth and increasing demand of water, deteriorating water quality, increasing environmental degradation and impeding climate change, more effort is required to assess water resources for national planning and management in order to sustain development.

Water resources have to be managed in an integrated manner, considering all the components of water cycle and all uses: agricultural, urban, rural and industrial, including the maintenance of the aquatic environment.

An approach that does not recognise and integrate these many dimensions of the system to the greatest extent possible can only produce an academic exercise at best. A more likely result will be a serious, perhaps irreversible, mismanagement of this vital source.

Efforts should be directed to overcome the present constraints regarding integrated water management approach and in particular, the institutional weakness, inadequate networks, incompatible techniques for field laboratory and office work. Deficiency of staff and their capability and the lack of coordinated, relevant research have constrained efforts further.

Challenges and opportunities for water resource development and management in the Mediterranean countries call for institutional reforms, improvement in the knowledge, capacity-building and internal cooperation on national, regional and global level.

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1. INTRODUCTION

The world is today-more than ever-subject to increased pressure on land water resources caused by population growth and industrial development.

Global water consumption has doubled in the 40 years from 1940-1980, and is expected to double again before the turn of the century. The majority of this consumption (some 70-80%) is for agricultural purposes, industrial activities account for some 20%, and domestic use for only about 6%.

The way in which water resources are being managed has increasingly severe environmental implications, including the accelerated soil and water degradation, the degradation of natural ecosystems and fresh water pollution.

Our attempts to develop and manipulate the earth for social and economic well-being have resulted in wide-ranging environmental damage that we are only now beginning to appreciate. Damage to the water components of the environment has resulted from deforestation, causing flooding, from agriculture causing the salinization and contamination of ground-water table with pesticides and fertilizers, and from industrial activities, causing toxic chemical contamination of our water supplies. We now realize that not only are our development activities affecting the water environment, but they are also jeopardizing our future use of water. This is of course the concept of sustainable development. The question we are now confronted with is: will this development lead to water crisis in many parts of the world, as predicted by some ?

The answer to this question is measured by our ability to improve the management of the world's water resources through cautions and sustainable development. Without a careful management of the available water resources, and without the adaptation of social demands to them, a sustainable development is not possible. This will be discussed in this paper, with special emphasis on the constraints and challenges in the developing countries, particularly those of the Mediterranean region.

2. THE WATER CRISIS IN THE MEDITERRANEAN

Water shortage is not a new phenomenon in the Mediterranean. What is new, however, is that it is occurring in an increasingly changing environment. The most recent drought in 1989 and 1990 summers marked a turning point that highlighted the vulnerability of water supplies even in the industrialized Northern Mediterranean countries which had always relied on an adequate capital of rainfall. The water crisis is endemic in some Southern Mediterranean areas, but it has now even reached some areas in France, Spain, Italy and Greece. The shortfall in quantity has been compounded by a decrease in quality due to the contamination of surface or underground water. In short, no country is safe from serious shortages of water, and management of water resources is one of the most urgent problems facing the Mediterranean basin.

There are many interrelated reasons which are contributed to this crisis. The major ones will be discussed herein:

2.1 Population Trends and Explosive Urban Growth

The population of the Mediterranean basin countries as a whole, currently being around 360 millions (Fig. 1), would reach between 520 and 570 millions by the year 2025. The Northern countries of the basin, from Spain to Greece, will account for only about one third of the total population, compared to two thirds in 1950 and about the half today. On the other hand, the countries South and East of the basin, from Morocco to Turkey, will contribute by nearly two thirds of the total Mediterranean population by the year 2025, i.e twice their current number and nearly five times what it was in 1950.

The sequences of this high population growth rate with an average of 3% yearly in the Southern countries of the Mediterranean will, as well, increase the total water requirements. Furthermore, past experience indicates that, as the standard of living increases, so does per capita water requirements.

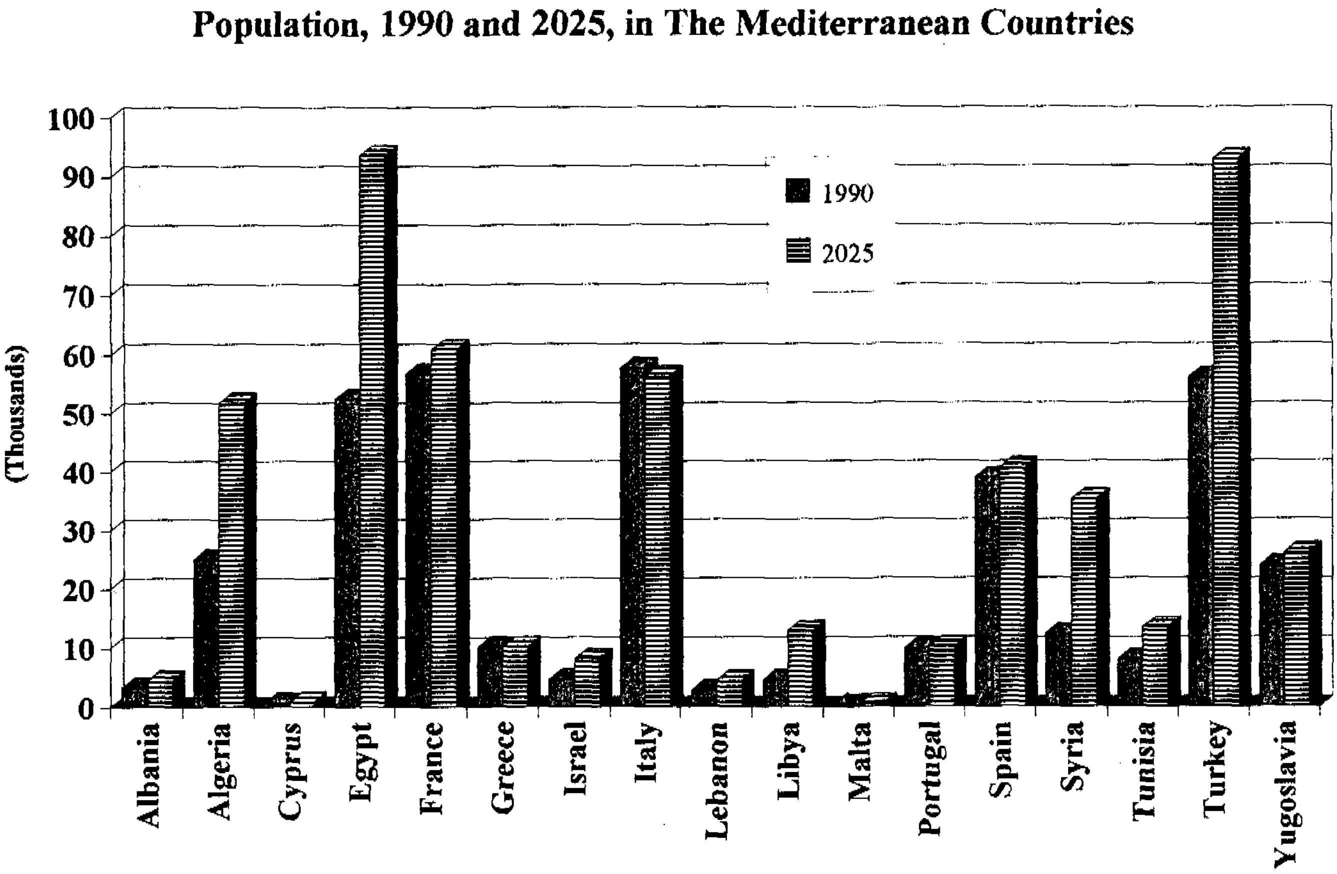

Figure 1. Population in the Mediterranean countries- Evolution trends 1950-2025 (source: Blue Plan, U.N).

Rapid population is always linked with a fast urbanization (Fig. 2). The size of urban population will be very large: 200 million more urban inhabitants in 2025 in the south and east of the basin, i.e, as much as the total urban population in the Mediterranean region at present. The urban population of the Mediterranean basin could, in fact, number between 380 and 440 millions compared to a little over 200 millions today. Generally, the annual growth of urbanization is high in the Mediterranean region, but it is much higher in the South (4.5%) with respect to the North (2.8%).

Urban and Rural population in the Mediterranean

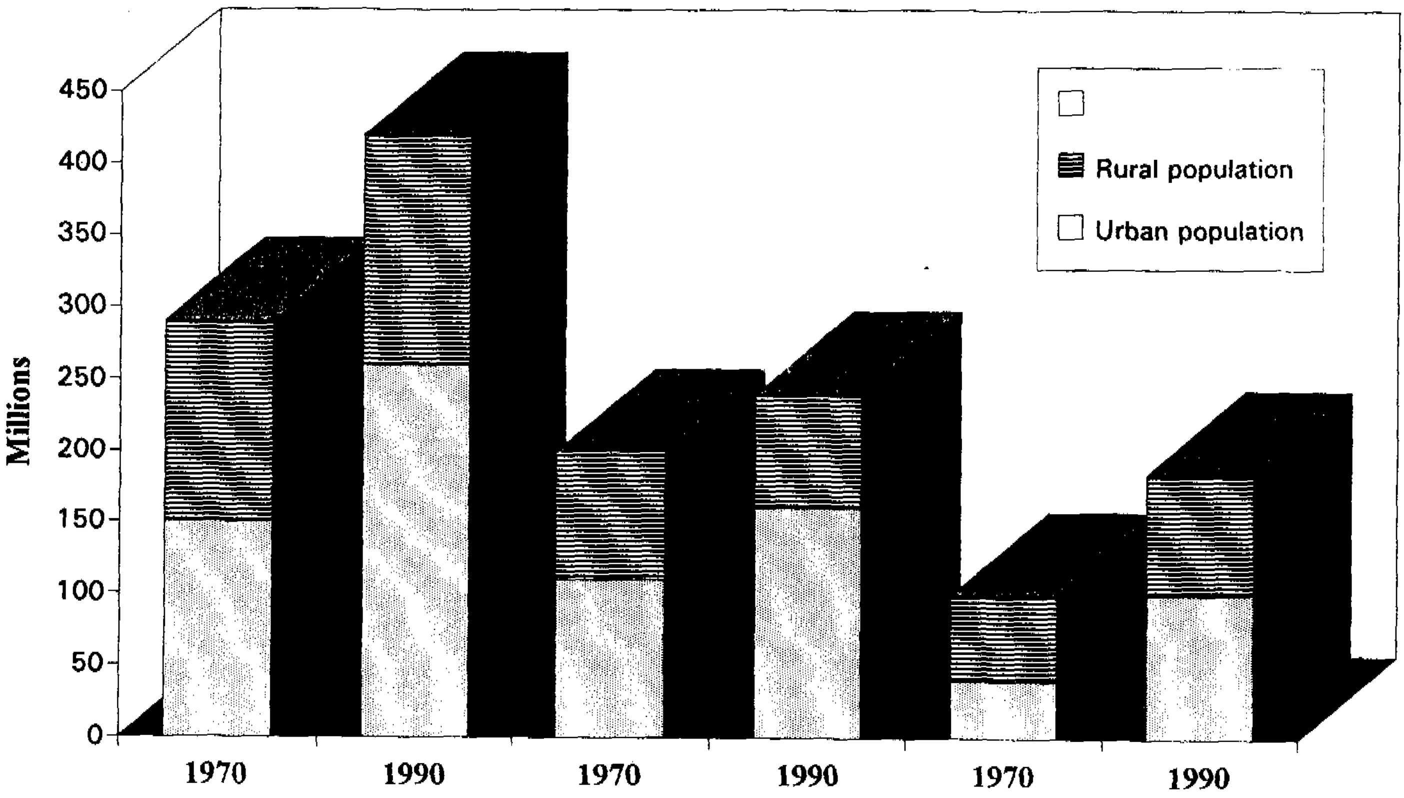

Figure 2. Urban and rural population in the Mediterranean

This population increase with high urbanization rate, will impose serious stress on the fresh water resources particularly on consumptive uses in the developing countries of the Mediterranean region. Southern and eastern Mediterranean countries will experience difficulties in ensuring self- sufficiency in meeting agricultural, domestic and industrial water needs. The supply of drinking water to urban areas will be one of the most critical problems in those countries.

2.2 Water Scarcity

In the southern arid and semi-arid countries of the Mediterranean, water is a resource which is scarce, limited, often of poor quality and vulnerable to sanitation and pollution, sometimes non-renewable, harmful to the soil and with devastating effect when flood occurs.

In the Mediterranean basin, as a whole, 72% of water resources is used for irrigation, 10% for drinking and 16% for industry. But sectorial water uses in the northern Mediterranean countries are completely different, compared to the southern ones (Fig. 3)

The overall picture is further complicated by other pressures on demand. Tourism on the coast during summer can double or triple withdrawals and lead to crisis situations. The presence of industries that are large consumers of water (power stations in industrialized countries around the Mediterranean, pulp factories such as the one in Mostaganem in Algeria, which withdraws 30 million m^3 annually) is another important factor. The development of urban centers around the basin means that the water supply can break-down when drought persists. Greece is facing this situation even though it is one of the richest countries from the point of view of water resources. Cairo, Algires and greater Tunis periodically face similar problems.

The Mediterranean countries are classified into three major groups with regard to future water problems:

1. Countries where available water supplies will remain relatively important up to and beyond 2025, allowing an increase in per capita withdrawals as a result of sustained efforts to develop and manage water supplies, in particular, to ensure suitable quality (France, Italy,Yugoslavia,Turkey, Lebanon, Albania);

2. Countries whose wafer resources are currently sufficient but will decrease, although these countries will be able to continue to meet their needs through water resource development provided that per capita withdrawals do not increase significantly (Spain, Morocco, Algeria, Cyprus);

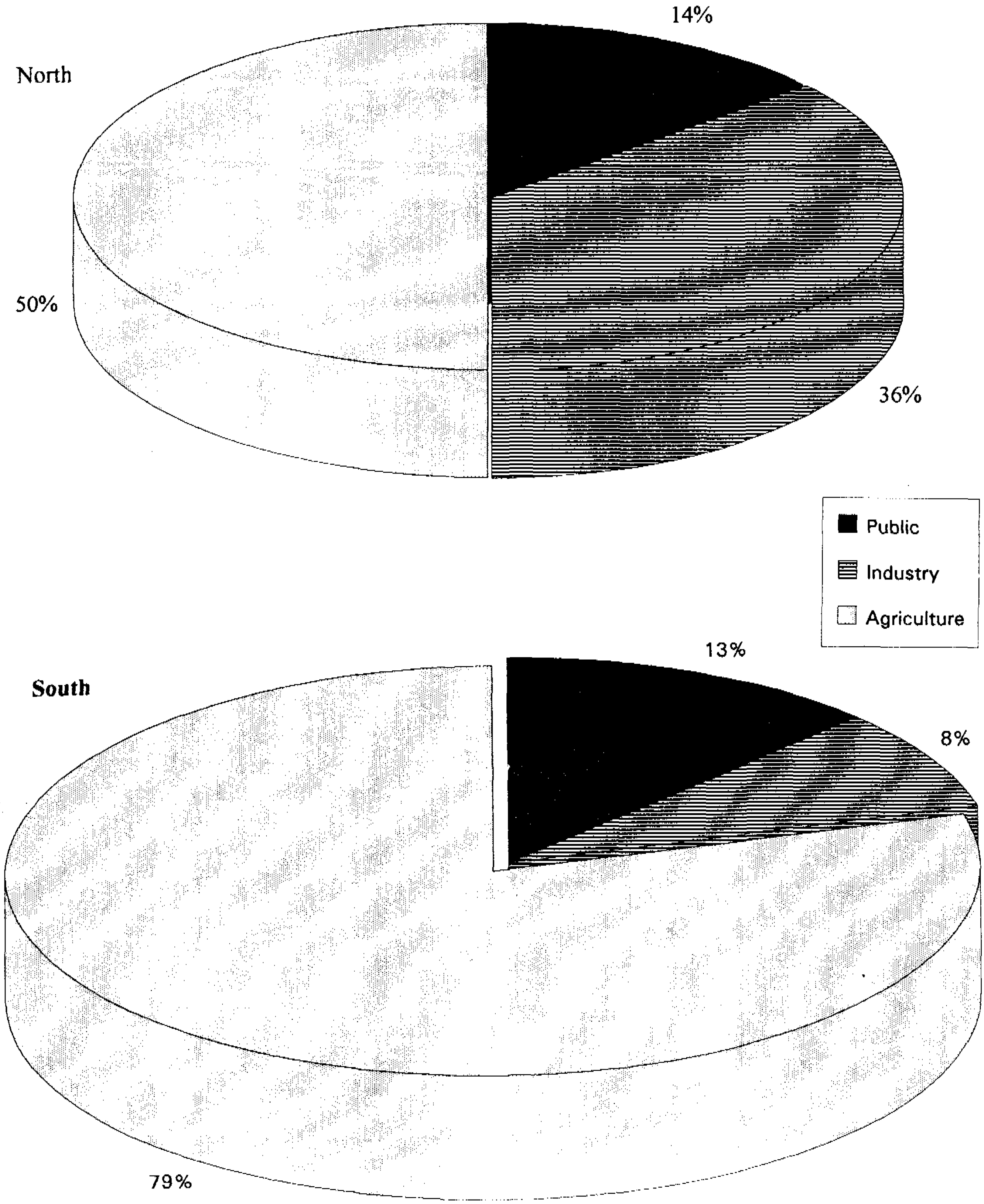

Figure 3.(a,b) Sectorial water use in the Mediterranean countries -year 1985- (source: elaboration LABMED on data of World Resources Institute, 1985).

3. Finally, countries whose water resources are already limited and which will have to make increased use of non-conventional resources (fossil water, desalination, imports), together with a reduction in per capita withdrawals (Malta, Egypt, Syria, Libya).

2.3 The Structural Imbalance

The dominant fact which will be strongly evident over the next few decades is the structural imbalance between the constantly increasing demand for water to meet the needs, and the natural available water resources.

Over the last few decades, the imbalance was limited to a few countries and requirements were met by the gradual additional harnessing of natural resources wrongly thought of as being infinite. This room for manoeuvre is progressively contracting and will be available less and less in the future.

In several Mediterranean countries, the imbalance will appear around the year 2000 and beyond. In the Southern Mediterranean countries, the water demands will fast approach the limit of resources and the majority of these countries could enter a period of chronic shortage during the nineties. These countries will be facing several similar problems that could be outlined in the followings (Hamdy and Lacirignola, 1992):

- Declining water resources per inhabitant both in terms of water availability and water withdrawals. It is expected that the available water/capita will be reduced by nearly 50% of the present one (Fig. 6).
- Exploitation of water at a relatively high rate with the risk of water quality deterioration.
- Excessive reduction in water withdrawals per capita, which will impose its significant effect on the water sectorial use, creating notable competition and conflict among users in the various sectors, in the irrigation and domestic sectors in particular. Priorities will be given to satisfy the drinking water demands to the expenses of the available water allocated for the irrigation sector with the consequence of less irrigated surface and more land degradation.
- Progressive degradation in the quality of available water resources because of increasing waste load discharged into water bodies and the atmosphere.

2.4 Water Quality Degradation and Water Pollution

In the Mediterranean developing countries, the water supply environment is sensitive and fragile. Industrial development is leading to severe over-exploitation of water resources, the pressure of urbanization, lack of understanding of the detrimental effects of the various forms of development and technology adopted had complex and degrading effects on the water resource quality. All the coastal expenses of water of the arid part of the Mediterranean are polluted or in process of being polluted by the sea-water intrusion and are becoming increasingly unsuitable for use.

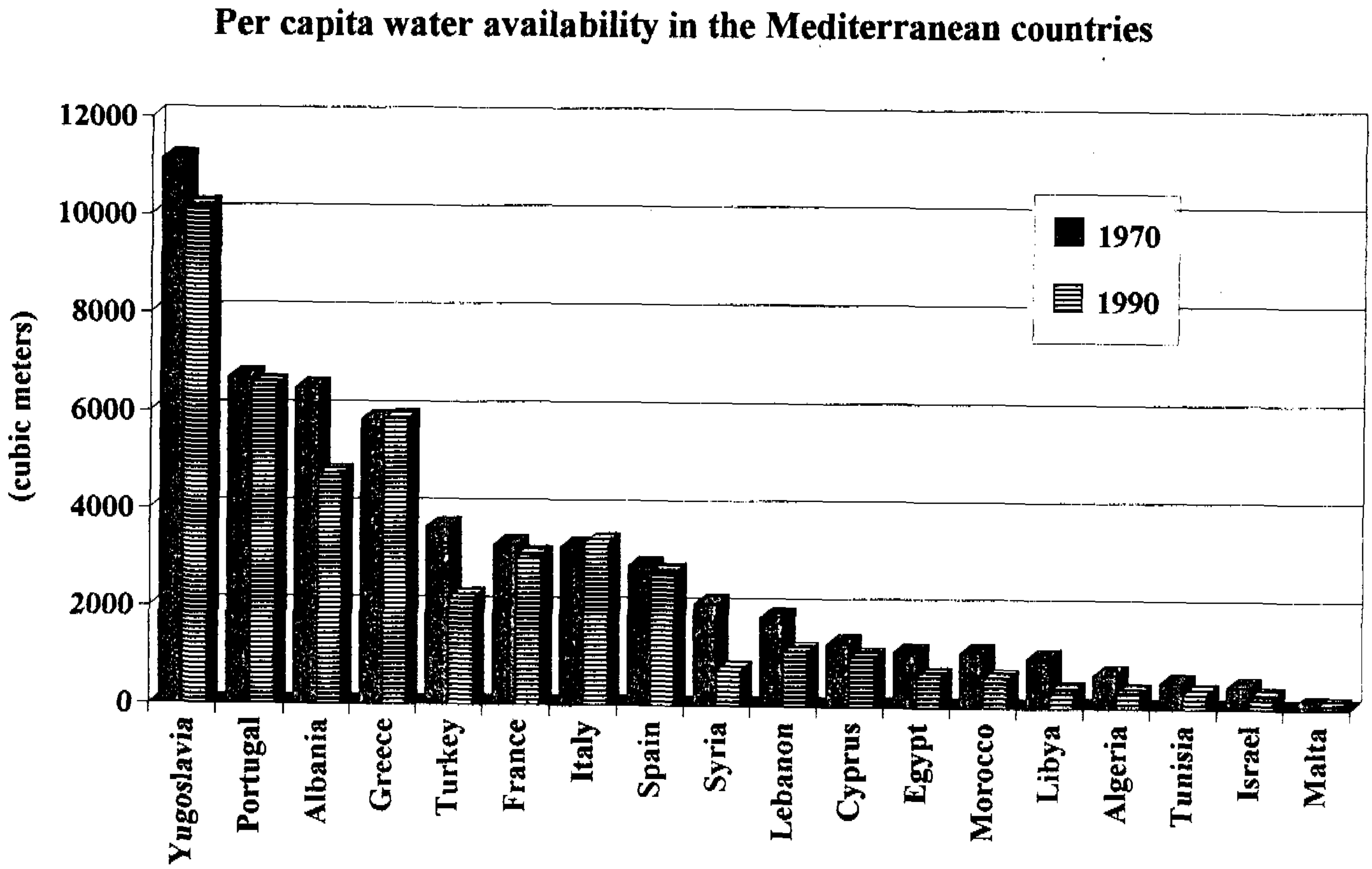

Figure 4. Water Availability per Capita in the Mediterranean Countries (source:World Resources Institute, 1985).

Human activities increase and more and more waste products are contaminating available water sources. Among the major contaminants are untreated or partially treated sewage, agricultural chemicals and industrial effluents. Many sources of water near the urban centers in the Mediterranean developing countries have been already contaminated, thus, impoverishing their potential use.

Agricultural activities which are very intensive in the southern and eastern parts of the Mediterranean, and in particular the use of both fertilizers and pesticides in huge quantities without the existence of real control and regulations for use, are major in the deterioration of water quality. The agriculture impacts of water quality (Biswas, 1992) could be outlined in the followings:

- Alternations in sediment load due to change in land use practices and cropping patterns;
- Water quality deterioration due to anthropogenic chemicals like fertilizers;

- Change in salinity and water logging;
- Water quality degradation due to effluents from agro-processing industries.

How can the problem be solved ?

As the population growth implies a higher per capita domestic and industrial demand as a result of the improved standard of living, the sustainable upper limit or "carrying capacity" of water resouces utilisation will be approached very rapidly over the next two or three decades in many countries. The traditional response of increasing water availability will be no longer adequate in the future for two main reasons:
i) Many countries simply do not have any major additional sources of water to develop economically.
ii) Even for those countries that may have additional sources of water, time periods required to implement those projects are likely to be much longer than expected at present.

The situation is most critical in countries which are heavily dependent on irrigation to meet their domestic food needs, unless planning methods are adopted which are designed to investigate water resources in a comprehensive manner.

In addition to the major constraints of the scarcity of water resources, particularly in the arid and semi-arid regions of the Mediterranean, the water resource managers are facing several problems concerning the environmental degradation, economic and financial constraints, ineffective institutions and weak capacity building. As to irrigated agriculture, the most dangerous problems that create unsustainability are soil erosion, high water table, and salinity and uneconomical production. The technical knowledge or capabilities exist to alleviate these problems, but they are not always affordable.

3. INTEGRATED WATER RESOURCES MANAGEMENT

Over the last twenty years, it has become clear that water resource management needs to broaden its focus. The problems water managers have to deal with include environmental, social, institutional and legal aspects.

In the context of the current critical water shortages, the statements of the Genoa Declaration of the Mediterranean Action Plan (1985), the declaration of Algires meeting on water (1990), the Nicosia Charter (1990) and its follow-up at the Cairo meeting (1992), the Action Programme adopted by the Mar del Plata Conference (1977), the declaration of International Conference on Water and Environment of Dublin (1992), the results of the World Congress on Environment and Development in Rio de Janeiro (1992) and the Mediterranean Charter for Water (Rome, 1992), all , support the integrated water management approach as a concept to alleviate water problems for a sustainable development.

Integrated water resources management has been adopted by various national governments and international organizations but each from their own perspective as the solution to a host problem. It should have the following characteristics:
- Interaction between quantity, quality and biological aspects for both ground-water and surface water;
- Sectorial coordination: water demands by different sectors of economy are considered in relation to the sectorial development and management plans, objectives and policies. Allocation of water resources should be consistent with the social and economic benefits of water utilisation in these sectors;
- Environmental sustainability;
- Capacity building: institutional and human resources development for the execution of management tasks;
- Implementation aspects, including financing, monitoring and control;
- Public participation considering social and cultural issues, as well as the traditional use of water.

The implementation of an integrated water resource management program to achieve the main goals of efficiency and equity requires further development in the following directions:

a) **Data collection and management:** data management is a critical information activity in a situation of water shortage. Data management must be integrated with regulation, water supply assurance, water allocation and planning for development.

Accurate, reliable and well-managed data should be a prequisite for successful water resource management.

b) **Demand management:** the principle that water is not a free goods, should be developed into an operational alternative for supply-oriented management. Water should be considered as an economic resource, and consequently adequate pricing should be implemented where applicable. The price of water should not only cover the direct costs of production, but should include its scarcity value as well. In the development of water pricing systems special attention should be paid to: (i)Pollution and over-exploitation (mining) in relation to long-term sustainable use of water; (ii)Social aspects of equity concerning the access to water and the ability to pay of low income groups.

c) **Public participation and community management:** Water resource development should pay more attention to the self-reliance of local communities, based on traditional approaches which are often acceptable and environmentally "sound". This requires public participation at all stages in the planning process. In addition, community management should be considered as a viable alternative to complement the necessary strong government coordination and planning.

d) Functional institutional arrangements: structure and linking mechanisms are crucial for implementing integrated water resource management, including these issues:

* coordination with other sectors of the economy and the overall economic development planning process,including the development and operation of cross-sectorial information systems;
* implementation of the planned actions, in particular in relation to demand management; and
* enforcement of regulations.

e) Careful planning and adequate supporting analysis: this is an important vehicle to prepare for successful implementation of integrated water resource management. Proper planning can improve decisions on investments and on allocation of water. Planning is not a blue-print exercise but should be an open-ended cyclical process. What is required at the national level is a capability for planning rather than cook-book master plans. A capability for planning would include:(i) the availability of any access to reliable and relevant data, including monitoring and control data on implemented measures; (ii) proper institutional arrangements and, (iii) adequate and well-trained staff.

f) Monitoring and control: this is important for both supply and demand management and should relate to both the water resource system and its users, with respect to water quality issues, such as source of pollution, distribution and accumulation in ecosystems, and harmonisation of sampling and analysis procedures. Without such monitoring and control, demand management is not likely to be successful.

4. MAJOR CHALLENGES

Although several important advances have been made over the last several years, significant challenges still remain in the areas of technological, managerial, policy innovation and adaptation, human resources development, information transfer and environmental considerations.

4.1 Water Conservation and Efficient Use of Water

In many parts of the Mediterranean, water conservation and efficient use of water have not been given the attention they deserve.

Since agriculture is by far the largest water user, efficient irrigation management will undoubtedly be a major conservation option in the future.

At present, it is fairly common to find that more than half the amount of water withdrawn from the resource does not even reach the fields being irrigated. In general, only about 25-30% of the water diverted into large canal systems in developing countries actually becomes available to the crops, leading to a world-wide irrigation efficiency less than 40%.

It is needed to find appropriate ways to achieve greater efficiency and equity in irrigation systems. Such an approach will help not only to achieve greater levels of agricultural production with lesser amounts of water, but also to address some of the world's major environmental problems- water logging and salinity, declining ground-water tables, shrinking lakes and seas whose root cause is over-watering. But finding such ways will require that a wider range of alternative approaches than heretofore considered will need to be developed, tested, and implemented, such as small-scale irrigation, conjunctive use, reuse of the unconventional water resources (Chambers, 1988). This will require much greater imagination and flexibility on the part of irrigation policy makers, managers and planners, and points to the need for technological, managerial, and policy innovation and adaptation. In particular, technologies, management practices, and policies that lead to greater control by end-users will be needed if the required increases in agriculture productivity are to be achieved. Procedures and practices for the assessment of the performance of irrigation at all levels must be improved with better management systems for water conveyance, allocation and distribution (IIMI, 1991).

4.2 Water Sectorial Use: Competition and Conflict

Any amelioration of conflict and competition among water users will have positive effects to improve efficiency and productivity. Greater efforts are urgently needed to integrate irrigation planning and management with other sectors of economy that impinge on water use (World Bank, 1991).

It is estimated that the allocation to agriculture will be reduced by 10-15% in the next 10-15 years, due to increased demand for house-hold and industrial needs. Therefore, it is essential to workout the trade-off analysis for water sectorial use in order to solve the water problem and any emerging crisis in the future irrigation development planning. Equally, by extending the use of optimization techniques to a wider audience concerned with water planning and management, including the complexity of the multiple demands now being made on the limited water resources and the far-reaching impacts which many water use activities are having.

4.3 Water Pricing and Cost Recovery

The most obvious reasons that make irrigation water pricing an issue of great importance in water management in the arid and semi-arid countries of the Mediterranean are that conceptually it could affect (Biswas, 1991):

- Water allocation between competing uses;
- Water conservation;

- Generation of additional revenue which could be used to operate and maintain water systems, and even repay part or all of investments costs;
- Cropping patterns;
- Income distribution;
- Efficiency of water management; and
- Overall environmental impacts.

There is an overall agreement now that a precondition for meaningful water resource management in the long-run is that water should be considered as an economic goods with an opportunity cost related to alternative future utilisation scenarios. However, the point still under argueing and discussion is on what criteria should the water charges be based? Should the beneficiaries pay the operation and maintenance costs of water systems? Or are they expected to pay the total investment costs as well? Should such pricing include external costs like environmental and social damage? If so, how should these costs be calculated?

These difficult issues are not easy to be resolved by policy and decision makers without extensive study and background information and a better understanding of the characteristics and motivation of the human components of the irrigation system.

4.4 Waste Water Reuse

Waste water reuse has always been an integral part of human life. In ancient times, it was practised on a small scale, thus, all adverse effects were considered as localized phenomena. But today, it is beyond doubt, effluent reuse is going to grow in much faster rate and scale than what we expected a decade ago. In view of this, all water reuse practices have to be viewed and analysed on a long-term and in the global context.

Lately reclaimed water use activities have been intensified. Unfortunately, these developments are not kept breast with creation of adequate sanitary regulations and effective enforcement agencies. This dichotomy has, in turn, created a spate of environmental and health hazards.

Even though, water reuse appears like a simple and appropriate technology, in reality, it is a complex one. It has multidisciplinary inter-linkage with different sectors such as: environment, health, industry, agriculture, water resources, etc. In addition, due to these complex interlinkages, in many countries, the administrative responsibility of reuse activities is not well defined, which further complicates the creation of regulation and its promulgation.

4.5 Water Quality Management

Water quality management will increasingly become important as water quantity management. Water quality monitoring will become essential for efficient water management.

Water quality management in developing countries is faced with numerous obstacles. It has been generally a neglected subject for various reasons among which are the lack of political will, resource and manpower constraints, institutional inertia and public apathy. In addition, water quality monitoring is a far more complex task than water quantity monitoring because there are a high number of parameters involved, higher costs in sampling and laboratory analysis, and higher requirements for skills and equipments.

For developing countries, it is important to determine water quality objectives and criteria. Standards should be directly adopted considering the country's social, economic, cultural and climatic requirements and the manpower expertise and institutions necessary to implement them. Furthermore, developing countries should realistically considere what can be achieved and then take appropriate actions to enforce it. Otherwise, best will continue to be the enemy of good.

4.6 Institutional Response to Better Management

A sectorial approach to water development is a major institutional constraint in all developed and developing countries. Water management can be rational only if the institutions responsible for such management are efficient.

In addition to institutional strengthening, nearly all countries have to substantially improve their inter-institutional collaboration in order to practise efficient water management policies in the future. At present, water related policies are developed in a fragmented fashion. For example, generally irrigation and large-scale water development come under Irrigation or Water Resource Ministry, domestic water supply under Ministry of Public Works, navigation under Ministry of Transport, hydropower under Ministry of Energy, environment impacts under Ministry of Environment and health issues under Ministry of Health. The co-ordination between these various ministries leaves much to be desired.

And yet in any large-scale water development project, all these issues must be integrated within the project area while it is easy to point out this necessity, how can this integration be effected in reality in the field is a very complex and daunting task.

4.7 Capacity Building

Capacity building in developing countries should be expanded and improved and interdisciplinary training of water experts should be promoted. To utilise water resources optimally, it is desirable to find and introduce new ways of interdisciplinary education and transfer of knowledge to developing countries. It should always be realised that traditional approaches as used in the developed countries may not be effective in finding solutions to problems in the developing countries (UNDP, 1992).

The growing body of national and international legislation on water, pollution, and the environment presents a challenge for future water managers, and requires broad training and exchange of professional information.

An important aspect of institutional arrangements is to create the capacity to implement effectively integrated water resource management. The capacity building efforts refer to the financial, administrative and technical capabilities of the institutions involved and a favourable policy environment.

WMO and UNESCO (1991) state that an important aspect of capacity building is the ability of a water authority to collect, analyse and elaborate information on water resources. This should include environmental and socio-economic information which is essential for integrated water resource management.

The need for education and training towards improving the water consciousness and water management ability of all nations should be promoted.

One important aspect of capacity-building is the supply of human resources. There is an urgent need for adequately trained professionals who can work in the multi-sectorial environment of integrated water resource management. In addition to the understanding of the technical disciplines related to the various water users, the future water resource managers should be knowledgeble about economics, ecology, and legal and social analysis in a far more dense and complex society.

Extensive educational programmes should be instituted at all levels in society to promote prudent use and conservation of water as one of the indispensable natural resources. Water consciousness at grass-roots level should be fostered through all stages of education to ensure self-help support of rural water shemes created by regional authorities, especially in developing countries. Linkages should be established with good health and domestic hygiene practices.

Here lies an important task for the Universities, national and international educational and training institutes to prepare the next generation of professionals for the immense tasks they are facing.

Tasks that are more complex than we can envisage today. One of the problems is that the teachers who have to educate the future generation make use of experiences gained in a less complex world. In addition, trained professionals should be able to work in an enabling environment with good career opportunities and incentive structures. If that is not taken care of, a costly brain-drain will follow.

5. CONCLUDING RECOMMENDATION

Water-related problems are increasingly in scale and intensity. Reduced quantities and deteriorating quality of available water result in an immediate reduced access to safe water for human activities as well as long-term environmental degradation.

Solving these problems either in developing or in developed countries is hampered by economic anf financial constraints. The problems are also compounded by inadequate and malfunctioning institutions at the national level, and insufficient coordination at the international level.

In the developing countries with limited available water resources, it is urgently needed to find viable and realistic water management strategies that can deal with the following four issues:
1- How to safeguard water to meet basic needs for difficult uses;
2- How to minimize water losses;
3- How to allocate scarce water for desired socio-economic development;
4- How to protect the environment from the degradation and less productive capacity.

The answer is an integrated management which should be concerned both with supply and demand, grounded on solid scientific and technical foundations and necessitating an interdisciplinary approach to the ecological, economic and social problems. Such management should aim at promoting the use of water resources in such a way to ensure the satisfaction of society's needs while preserving them for the future.

For successful implementation of integrated water management approach to achieve its main goals; equity and efficiency, concreted actions are required at the international and local levels. Demand management and the corresponding institutional changes are high priority actions which essentially belong to national and/or local responsibilities. The international organizations play an important role in the development and implementation of international rules and legislation; research and technological developments for more efficient water use, education, training and capacity building and awareness and promotion.

We need to rethink our whole approach to water. Efficiency must be the option of first choice. Efficient irrigation management will undoubtly be a major conservation option for the future

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STOCHASTIC MODELING OF THE NILE RIVER FLOWS

Jose D. Salas[1], Magdy W. AbdelMohsen[2], and Bayoumi B. Attia[3]

Stochastic modeling of hydrologic time series is important in analyzing water resources systems. Historically, modeling the Nile River flows has been a challenging problem in stochastic hydrology because of its complex long term dependence structure. In this paper, a new model called multiplicative PARMA (XPARMA), for generating seasonal flows of the Nile River is proposed which is capable of preserving seasonal and annual variance-covariance statistical properties. Computer experiments showed that the new model is a favorable alternative for generating monthly flows of the Nile River.

Résumé

La modélisation stochastique des séries chronologiques des débits est importante pour l'analyse des systemes hydriques. La modélisation stochastique dés debits du Nil a posé un défi d'envergure à cause de la structure de dépendance à long terme. Dans cet article, un nouveau modèle appelé "PARMA multiplicatif" (XPARMA) est proposé pour la génération des débits saisonniers, lequel modele est capable de préserver les covariances saisonnières et annuelles. Des essais montrent que le nouveau modèle constitue un choix valable pour la génération des apports mensuels du Nil.

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1. INTRODUCTION

Stochastic modeling of hydrologic time series has been widely used for planning and management of water resource systems such as, reservoir sizing, testing reservoir operating rules, and determining the risk of failure (or reliability) associated with water resources projects (Salas et al., 1980; Bras and Rodriguez, 1985; Vogel and Stedinger, 1988; Freveret et al., 1989; Salas, 1993; Shahin et al., 1993). Stochastic models are used in operational hydrology to generate synthetic time series which exhibit similar statistical characteristics as the observed data. Such synthetic series are then used in the design and management of water resources projects in order to incorporate the effect of uncertain hydrologic data and be able to estimate the probability distribution of key decision parameters such as the storage size, maximum flow periods, or longest droughts (McLeod and Hipel, 1978; Martin and Julian, 1979; Bras and Rodriguez-Iturbe, 1985; Colosimo et al., 1988; Yen, 1988).

A number of stochastic models has been suggested in the literature for synthetic generation and forecasting of hydrologic processes (Bras and Rodriguez-Iturbe, 1985; Salas, 1993). Hydrologic processes such as, annual streamflow or annual precipitation, that fulfill the assumption of stationarity can be well represented by stationary autoregressive (AR) or autoregressive moving average (ARMA) models. These models have been well described in the statistical literature (see for instance, Box and Jenkins, 1976 and Brockwell and Davis, 1991). Stationary models applied to annual hydrologic processes are usually capable of preserving the historical annual statistics such as, the mean, the variance, and the covariance. In addition, ARMA models, may preserve long-term statistics such as the storage and drought related statistics (O'Connell, 1974; Hipel and McLeod, 1978; Salas et al, 1979).

However, in the case of monthly flows or monthly precipitation, the mean, variance, and the covariance depend on the season, and for this reason stationary models such as, AR and ARMA models, generally do not apply. Periodic autoregressive (PAR) models were introduced and developed in the 1950's and 1960's for modeling such seasonal processes (Hannan, 1955; Thomas and Fiering, 1962; and Yevjevich, 1972). These traditional periodic models are capable of preserving periodic statistics such as the seasonal means, standard deviations, and season to season correlations, but usually they fail to reproduce annual statistics such as, the annual covariance, and drought and storage related statistics. The preservation of both seasonal and annual statistics is essential for many water resources projects and it is difficult to achieve with PAR models.

The disaggregation model (Valencia and Schaake, 1974) has been developed to overcome the problem of reproducing both the seasonal and the annual statistics. In this model, the annual time series is first generated by means of a suitable stationary model, then the generated annual series are disaggregated into seasonal series such that the seasonal statistics are preserved. The main drawback of such model is the need to estimate a large number of parameters. Several attempts have been made in the 1970's and 1980's intended to reduce the number of parameters involved in disaggregation models (Lane, 1979; Salas et al, 1980; Stedinger et al, 1985; Grygier and Stedinger, 1988; Santos and Salas, 1992).

Periodic autoregressive moving average (PARMA) models are an outgrowth of PAR and ARMA models. Just like ARMA models are more flexible than AR models for modeling annual hydrologic series, PARMA models are more versatile and suitable than PAR models for modeling periodic processes such as monthly streamflow (Salas et al, 1980). Thus, in many cases PARMA models may be able to preserve both seasonal and annual statistics without resorting to disaggregation. In addition, PARMA models are useful for modeling hydrologic processes at different time scales in such a way as to achieve model compatibility at all scales (Obeysekera and Salas, 1986; Bartolini and Salas, 1993). However, in complex cases even PARMA models may not be suitable.

The main purpose of this paper is to present a stochastic model which is capable of reproducing not only the seasonal statistics but also the annual statistics. This can be accomplished by modeling the seasonal series directly and does not require disaggregation. The proposed model will be illustrated by using the monthly flows of the Nile River at Aswan station.

2. PREVIOUS STUDIES ON THE NILE SYSTEM

In the planning studies for the design of the Aswan Dam, Hurst (1951) realized that the use of only the available historical streamflows of the Nile River may lead to unreliable results regarding the storage capacity of the reservoir. Hurst then used probability theory and statistical experiments to derive expressions for the expected conditional range of independent random variables and for the expected rescaled range of geophysical variables which led to much controversy in stochastic hydrology for several decades and to the so called Hurst phenomenon. Thus, inspired by Hurst studies on the Nile River, a number of stochastic models were applied or were specifically developed , such as the Fractional Gaussian Noise (Mandelbrott and Wallis, 1969), the family of AR and ARMA (Yevjevich, 1963; Yevjevich, 1967; O'Connell, 1974; Hipel and McLeod, 1978), the Broken Line (Mejia et al, 1972; Curry and Bras, 1978), and the shifting level (Boes and Salas, 1978) models, for modeling annual streamflow data.

Curry and Bras (1978) applied the multivariate PAR(1) model and the multivariate broken line/disaggregation (BL/D) modeling scheme to generate monthly flows of the Nile River system. The PAR(1) model simulates monthly flows directly, while the BL/D scheme simulates annual flows first which subsequently are disaggregated into monthly flows. Both models performed reasonably well in reproducing the monthly means and standard deviations except for the Mongalla station for which the PAR(1) model underestimated such monthly statistics in the order of 10% and 20%, respectively. On the other hand, the PAR(1) model reproduced the monthly skewnesses and the month-to-month correlations better than the BL/D scheme. Regarding monthly lag-zero cross-correlations, both models performed quite well. In addition, the BL/D scheme showed a more consistent reproduction of the annual statistics than the PAR(1) model.

A comprehensive stochastic modeling study of the Nile River system was undertaken in the Master Plan for Water Resources Development and Use, Egypt (1981). The study was intended to identify a suitable stochastic model to generate seasonal inflows to Lake Nasser. Five multivariate stochastic models were included in the study, namely: (1) PAR(1) model with normal distribution, (2) PAR(1) model with log-normal distribution, (3) BL/D modeling scheme, (4) ARMA(1,1) model with disaggregation to seasonal flows, and (5) AR(1) model with correlated noise. The latter model considers the relation of the seasonal flows of a given year with those of the previous year and the correlated noise was modeled by an AR(1) process. Thus, in principle the model has the capability of preserving not only seasonal statistics but some annual statistics as well. Two versions of the model were developed, the first version was intended to preserve low order correlations, while the second version was fitted to preserve high order correlations. Further description of this model can be found in the paper by Franchini et al (1985).

The model selection criteria was based on the preservation of basic streamflows statistics such as, the mean, variance, skewness, lagged autocorrelations, Hurst slope, and the probability distribution of flows, and on several measures of reservoir performance. In applying the above mentioned (five) models to the Nile River monthly flows at Aswan station, a reduction of 8% was made to the flows prior to the year 1902 in order to make the time series of the record starting in 1871 stationary (the study considered that the change in the Nile River flows around 1902 was primarily attributed to a change in the tropical rainfall pattern about that time and was only partially explained by a change in the rating curve following the construction of the old Aswan Dam). The Master Plan study concluded that none of the models tested to generate seasonal flows in the Nile River system was built to preserve both seasonal and annual statistics without the use of disaggregation, and the problem with modeling schemes, which are based on disaggregation, is the large number of parameters involved. Even model 5, although is not based on disaggregation, has the drawback of the large number of parameters. The study recommended that "a method for preserving both annual and inter-month correlations within a stochastic model should be developed to have a better performance than the disaggregation techniques so far developed, but with a fewer number of parameters". The model described in the following sections of this paper is a step in this direction.

A detailed description of the Nile River basin, its various hydrologic characteristics related to rainfall, evapotranspiration, runoff, groundwater, water levels, water storage and conservation, and its basic streamflow time series statistics were documented by Shahin (1985). Also, Shahin (1990) studied the variations of the Nile flows and considered the period 1912 to 1960 to be fairly stable in regard to hydrological changes. For instance, during that period the maximum and minimum annual levels of Lake Victoria varied within limited ranges. This relatively stable period made possible the application of simple regression models for flood forecasting (Shahin, 1990). In addition, Shahin documented that since 1960, there are significant fluctuations in Lake Victoria levels, and reported the existence of long-term periodicities of 21, 7, 6, 4.2 and 2.7 years in the Nile flows at Aswan, which has been obtained by using correlation analysis. Moreover, the annual minima and maxima levels of the Nile at Roda, opposite to Cairo, which have been recorded since the year 622 AD, have been fitted by autoregressive models of order 13 for the minima levels, and order 17 for the maxima levels (Shahin et al., 1993).

Reusing and Skala (1990), in studying droughts in the Nile River system, performed the statistical analysis, modeling, and generation of the Nile River time series of maximums and minimums stage levels (after corrections for inconsistencies and after trend removal), based on the period of record 715-1469/70 A.D. The ARMA and FGN models were applied, 10,000 values were generated from each model, and the distribution of drought properties such as, duration, magnitude, and severity were obtained and compared with those derived from the historical data. The probability distribution of drought durations were used to determine the expected drought durations for different return periods. Their results showed that the FGN model gave greater expected drought durations than the ARMA model did. The results obtained from the stage level analysis were applied and compared to evaluate the droughts to be expected regarding annual streamflow in the Nile.

3. MODEL DESCRIPTION

The stationary autoregressive moving average ARMA (p,q) model can be written as (Box and Jenkins, 1976; Brockwell and Davis, 1991)

$$\phi(B)\ x_t = \theta(B)\ \varepsilon_t \qquad (1)$$

where p is the autoregression order and q is the moving average order; x_t is the a stationary normally distributed process with mean zero at time t, ε_t is normally distributed uncorrelated random noise with mean zero and variance $\sigma^2(\varepsilon)$ and $\phi(B)$ and $\theta(B)$ are operators defined as

$$\phi(B) = 1 - \sum_{i=1}^{p} \phi_i\ B^i \qquad (2a)$$

$$\theta(B) = 1 - \sum_{j=1}^{q} \theta_j\ B^j \qquad (2b)$$

in which B is the backward operator such that $B^k\ z_t = z_{t-k}$; ϕ_i, i=1,2,..,p and θ_j, j=1,2,..q are the autoregressive and moving average parameters, respectively.

Likewise, the periodic ARMA(p,q) model is defined as (Salas et al., 1980)

$$\phi_\tau(B)\ y_{v,\tau} = \theta_\tau(B)\ \varepsilon_{v,\tau} \qquad (3)$$

where $y_{v,\tau}$ is a periodic correlated normally distributed process with mean zero, $\varepsilon_{v,\tau}$ is an uncorrelated normal process with mean zero and variance $\sigma_\tau^2(\varepsilon)$, and $\phi_\tau(B)$ and $\theta_\tau(B)$ are operators defined as

$$\phi_\tau(B) = 1 - \sum_{i=1}^{p} \phi_{i,\tau} B^i \qquad (4a)$$

$$\theta_\tau(B) = 1 - \sum_{j=1}^{q} \theta_{j,\tau} B^j \qquad (4b)$$

with $B^j Z_{v,\tau} = Z_{v,\tau-j}$ for $j \le \tau$, otherwise $B^j z_{v,\tau} = z_{v-1,\omega+\tau-j}$; $\phi_{i,\tau}$, and $\theta_{j,\tau}$ are the periodic autoregressive and moving average parameters, respectively, v = year, τ = season ($\tau = 1,\ldots,\omega$) and ω is the number of seasons per year. The formulation (3) allows the model's parameters to vary with the season. Thus, enabling the model to preserve the seasonal statistics.

Box and Jenkins (1976) introduced a multiplicative model for periodic series. Consider the process z_t with period ω, and the model

$$\Phi(B^\omega)\ z_t = \Theta(B^\omega)\ \xi_t \qquad (5)$$

where

$$\Phi(B^\omega) = 1 - \sum_{i=1}^{P} \Phi_i B^{i\omega} \qquad (6a)$$

and

$$\Theta(B^\omega) = 1 - \sum_{j=1}^{P} \Theta_j B^{j\omega} \qquad (6b)$$

Model (5) can preserve the correlations between $z_t, z_{t+\omega}, \ldots, z_{t+P\omega}$. However, the process ξ_t is not uncorrelated because z_t can be correlated with $z_{t+1}, z_{t+2}, \ldots, z_{t+p}$ as well. This can be fixed by fitting an ARMA(p,q) model to the process ξ_t as

$$\phi(B)\ \xi_t = \theta(B)\ \varepsilon_t \qquad (7)$$

where ε_t is now an uncorrelated random process. Combining (5) and (7) one gets the multiplicative ARMA(p,q)(P,Q)$_\omega$ model(Box and Jenkins, 1976) as

$$\phi(B)\ \Phi(B^\omega)\ z_t = \theta(B)\ \Theta(B^\omega)\ \varepsilon_t \qquad (8)$$

The drawback of model (8) is that the model parameters do not depend on the season which results in the inability to preserve the seasonal correlations. One can modify this formulation by making the model parameters depend on the season, similar to the periodic ARMA model. Thus, the multiplicative periodic ARMA model (XPARMA) is

$$\phi_\tau(B)\ \Phi_\tau(B^\omega)\ y_{v,\tau} = \theta_\tau(B)\ \Theta_\tau(B^\omega)\ \varepsilon_{v,\tau} \tag{9}$$

For instance, for p=q=P=Q=1, the XPARMA(1,1)(1,1)$_\omega$ model is

$$\begin{aligned} y_{v,\tau} = {} & \Phi_{1,\tau}\ y_{v-1,\tau} + \phi_{1,\tau}\ y_{v,\tau-1} - \Phi_{1,\tau}\ \phi_{1,\tau}\ y_{v-1,\tau-1} \\ & + \varepsilon_{v,\tau} - \Theta_{1,\tau}\ \varepsilon_{v-1,\tau} - \theta_{1,\tau}\ \varepsilon_{v,\tau-1} + \Theta_{1,\tau}\ \theta_{1,\tau}\ \varepsilon_{v-1,\tau-1} \end{aligned} \tag{10}$$

In this formulation the model can preserve the annual covariances for each season τ in addition to the season to season covariances. The preservation of the annual covariances for each season is expected to improve the capability of preserving the annual covariances of the corresponding annual series.

The properties of the general XPARMA model are difficult to derive analytically. Salas and AbdelMohsen (1991) derived the properties of the XPARMA(1,1)(1,1)$_\omega$ model and showed that it is possible to preserve the annual correlations for each season by a suitable choice of the orders P and Q of the parameters $\Phi_{i,\tau}$ and $\Theta_{j,\tau}$, while the parameters $\phi_{i,\tau}$ and $\theta_{j,\tau}$ are only able to control the season to season correlations.

To estimate the parameters of the XPARMA model, the least squares method have been applied so as to minimize the objective function

$$F = \sum_{v=1}^{N} \sum_{\tau=1}^{\omega} \varepsilon_{v,\tau}^2$$

where N is the number of years of the observed data.

4. APPLICATION TO THE NILE RIVER FLOWS AT ASWAN

The proposed XPARMA model has been applied to the monthly flows of the Nile River at Aswan station. The Nile River is the major source of water supply in Egypt and the management and distribution of its flows are very vital for the development and improvement of water use in Egypt. Synthetic traces generated from the model can be utilized for future planning studies in the Nile River system, for testing new reservoir operating rules, and for forecasting future flows. The application study presented here is limited to the use of the XPARMA model for data generation.

Monthly and annual flows of the Nile River for the period 1871-1989 have been analyzed. The first 30 years of record appears to be a period of high flows. Some past studies adjusted the flows of such initial high period (Master Plan, 1981) prior to analyzing and modeling the flow data, others deleted the initial period, while others kept the total record. In the study presented here the total record was utilized. Figure 1(a) shows the historical annnual flows for the period considered in the analysis, and Fig. 2(a) shows the corresponding monthly flows for the first twenty years of record. The original monthly flow data showed monthly skewness coefficients significantly greater than zero for most of the months, thus the logarithmic transformation was applied in order to lower the skewness and to make the data approximately normal.

Five periodic models have been fitted to the observed transformed monthly flows. They are: PAR(1), PARMA(1,1), PARMA(2,1), XPARMA$(1,1)(1,1)_{12}$, and XPARMA$(1,1)(3,0)_{12}$. To verify and validate the fitted models, five hundred series of the same length as that of the observed flows (119 years) were generated from each model and the mean and the standard deviation of their monthly and annual statistics were computed. As expected the monthly means and monthly standard deviations are well reproduced by all models. The comparison of the month to month correlations show that all models reproduce these correlations quite well for small time lags, but significant differences exist between the models for longer time lag correlations. For instance, Figs. 3 (a) and (b) show the lag-1 month to month correlations obtained based on the historical and the generated samples from the PARMA(1,1) and XPARMA$(1,1)(1,1)_{12}$ models. While both models show that such lag-1 correlations are well reproduced, significant differences between the models occur for longer time lags. For instance, the same figure shows that for lag-48 correlations, the PARMA(1,1) model yields practically zero correlations for all months, while the XPARMA$(1,1)(1,1)_{12}$ model yields (above zero) correlations comparable to the historical ones.

Another way of comparing the historical and generated correlations is by calculating them for each month but with time lags which are multiples of 12, they are also called annual correlations for each month. For example, Figs. 4(a) and 4(b) show the annual correlograms for the April and June flows, respectively, determined from the historical record and from the flows generated by the various models. One may observe that only the multiplicative PARMA models are capable of producing long term correlations such as those of the historical record.

An attractive feature of the proposed XPARMA model is its ability to capture not only (short and long time lags) month to month correlations, but also correlations of the aggregated annual flows. Figure 5 displays the correlograms of the historical and generated annual flows for all the models tested (note that the referred models are those corresponding to the monthly flows). The figure shows that the PAR(1), PARMA(1,1), and PARMA(2,1) models fail to capture the long term dependence structure of the annual flows. In fact, for these models, the correlations after lag 1 are practically zero. On the other hand, the annual correlations derived from the XPARMA$(1,1)(1,1)_{12}$ and XPARMA$(1,1)(3,0)_{12}$ models, resemble the historical ones quite well, and the first model appears to represent such annual correlations somewhat better than the second. In terms of model parsimony, the XPARMA$(1,1)(1,1)_{12}$ model would be preferred over the XPARMA$(1,1)(3,0)_{12}$ model.

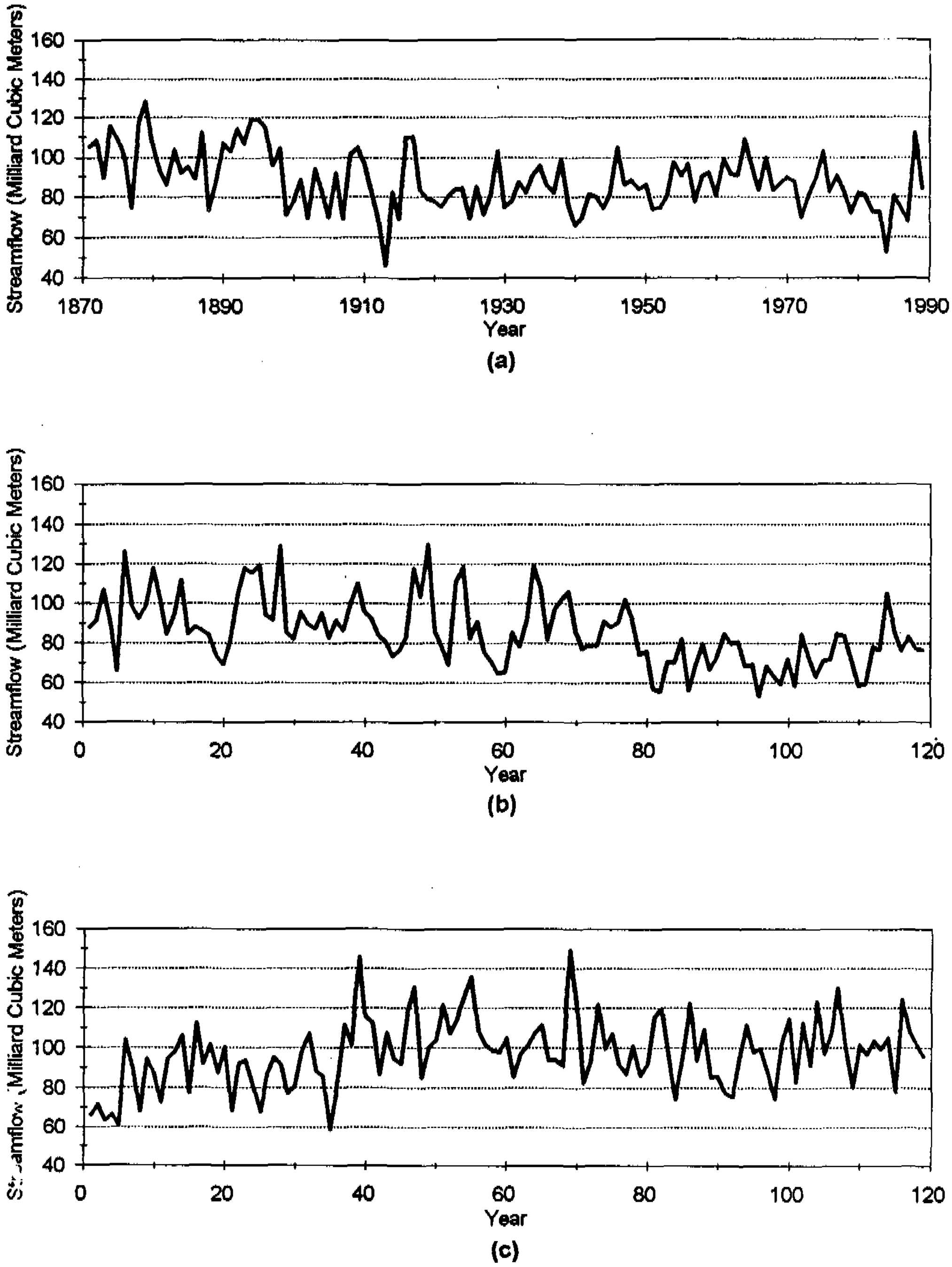


Figure 1 Historical and Generated Annual Flows
(a) Historical Annual Flows (1871-1989)
(b) Generated Annual Flows 1 (119-Years)
(c) Generated Annual Flows 2 (119-Years)

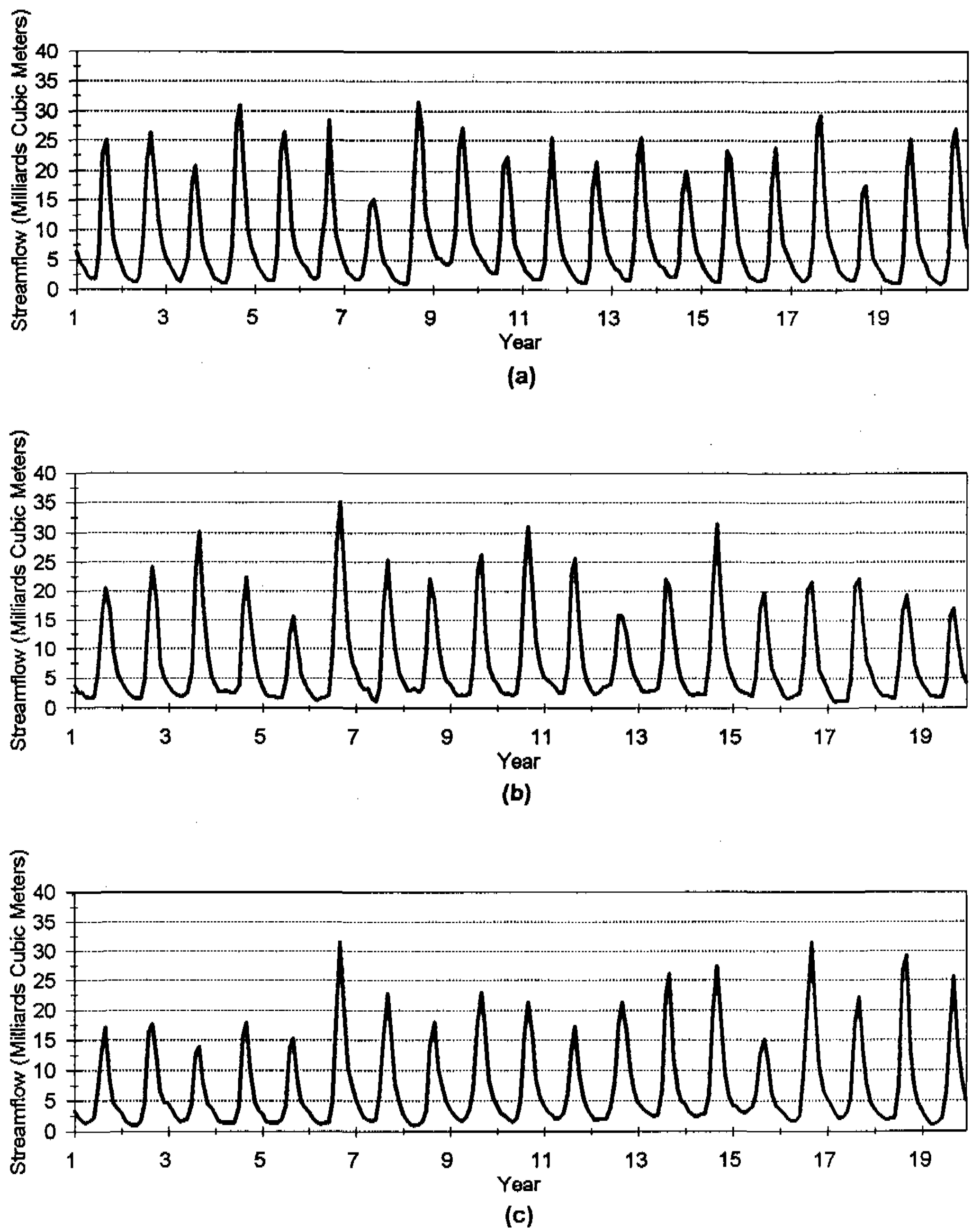
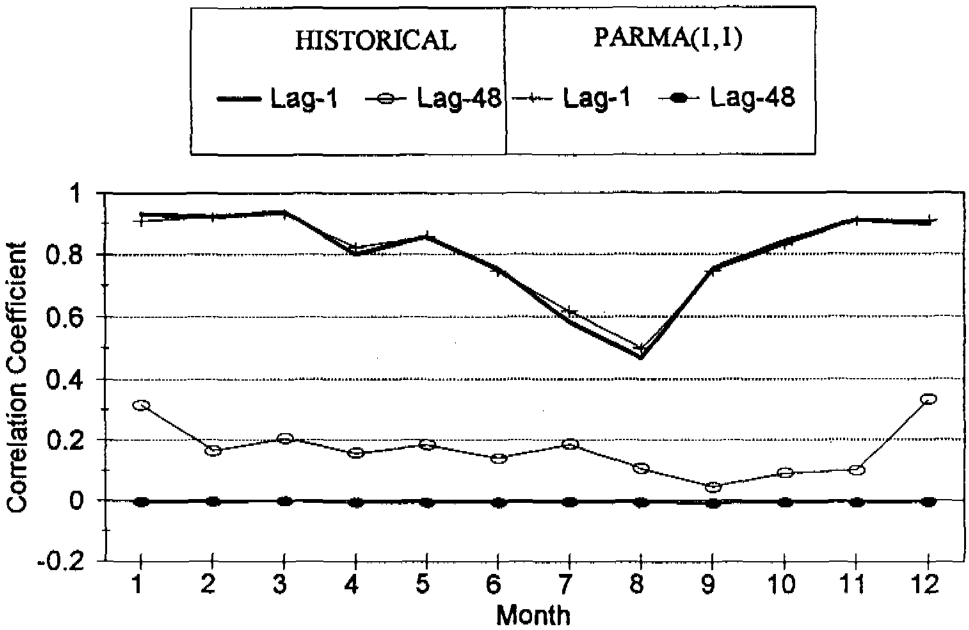
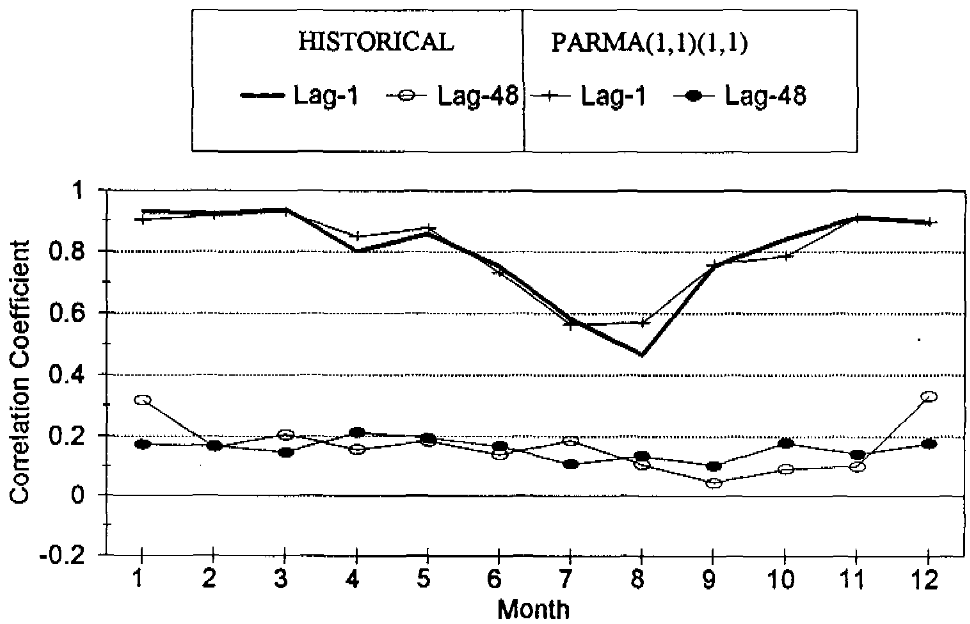


Figure 2 Historical and Generated Monthly Flows
(a) Historical Monthly Flows (1871-1890)
(b) Generated Monthly Flows 1 (20-Years)
(c) Generated Monthly Flows 2 (20-Years)

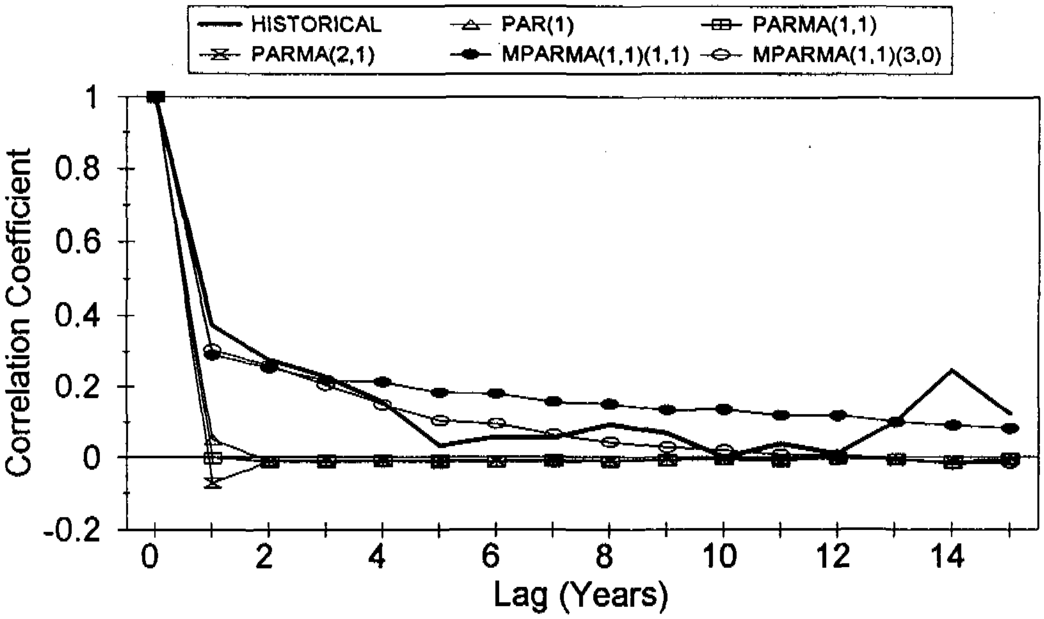


(a)

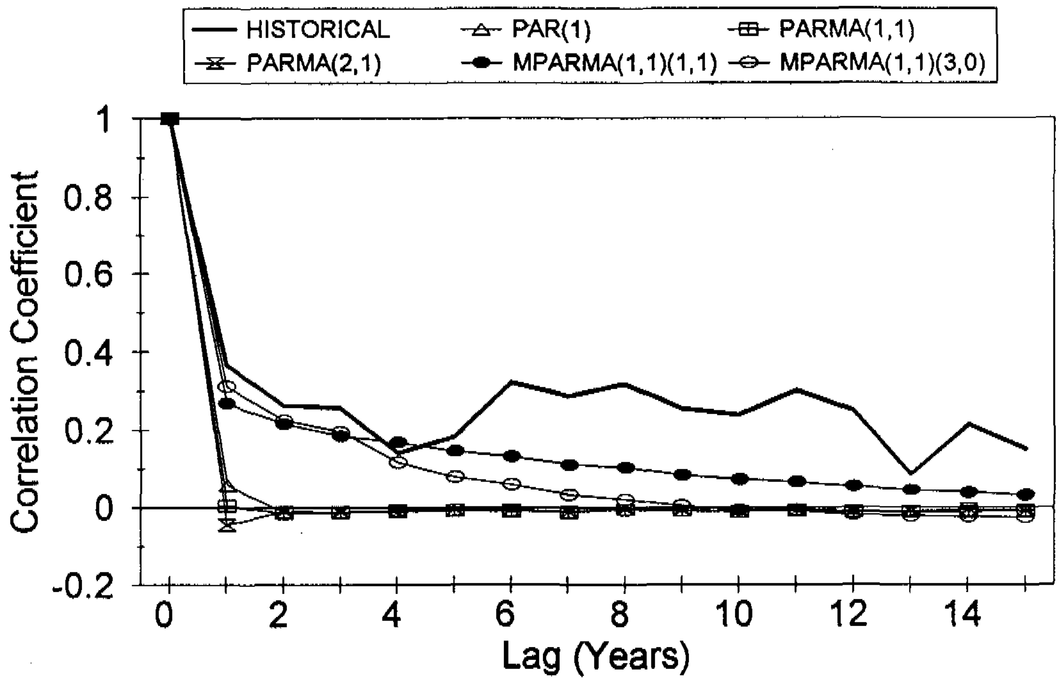


(b)

Figure 3 Historical and Generated Month to Month Correlations
(a) Historical and PARMA(1,1)
(b) Historical and XPARMA(1,1) (1,1)



(a)



(b)

Figure 4 Historical and Generated Annual Correlations
(a) Annual Correlograms for April Flows
(b) Annual Correlograms for June Flows

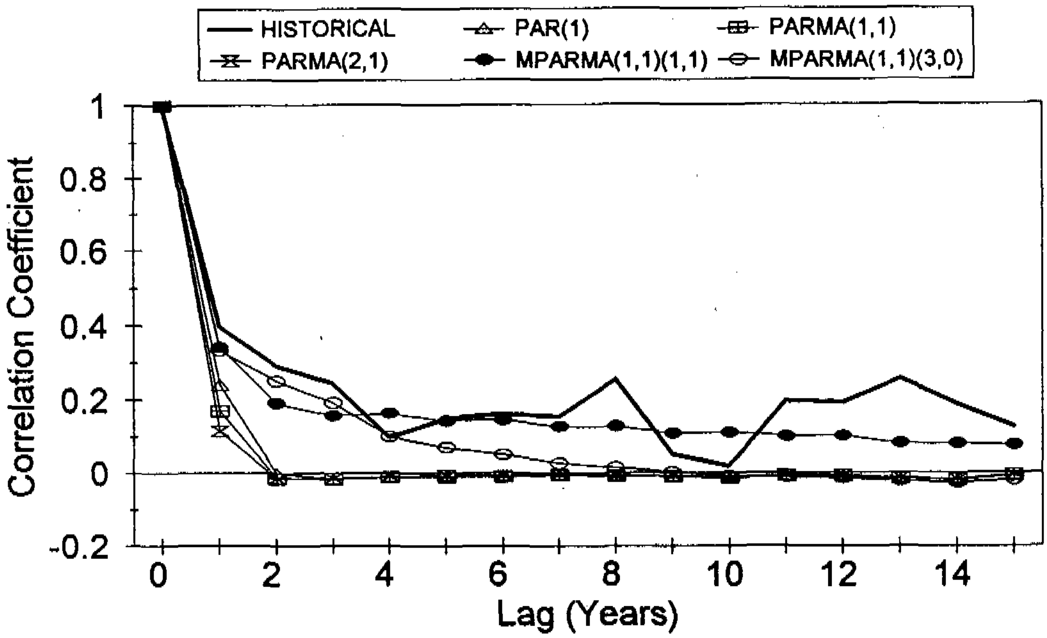


Figure 5 Annual Correlograms for Annual Flows

Other annual statistics besides the annual correlations were compared. They include the mean, standard deviation, skewness coefficient, longest drought, maximum deficit, adjusted range, and the Hurst coefficient. The longest drought and maximum deficit were determined relative to the sample mean. Table 1 gives the historical statistics and those obtained from data generation. For each model, the mean and the standard deviation of each referred statistic were derived based on the 500 generated samples. Table 1 gives the results obtained. The main difference seen between the models are those corresponding to the drought and storage related statistics. These statistics are underestimated by the PAR(1), PARMA(1,1), and PARMA(2,1) models, while they are well reproduced by the multiplicative models.

Figures 1(b) and 1(c) show plots of two samples of the annual flows obtained from the generated monthly flows based on the fitted XPARMA$(1,1)(1,1)_{12}$ model. The figures show that the pattern of the generated annual flows are similar to the pattern observed in the historical record, i.e. the variability is similar and the apparent shifts are observed in both the historical and the generated samples. The capability of the XPARMA$(1,1)(1,1)_{12}$ model to reproduce such complex flow pattern is the major factor in the ability of such multiplicative model to reproduce long-term related statistics such as the longest drought and the Hurst coefficient. In addition, Figs. 2(b) and (c) show two sequences of twenty years of generated monthly flows (they correspond to the annual sequences shown in Fig. 1). They show that the XPARMA model also reproduces the periodicity and the variability shown by the historical monthly flows.

Table 1 Comparison of generated and historical annual statistics for Nile River Flows at Aswan

Models	Mean 10^9m^3	Standard Deviation 10^9m^3	Skewness Coefficient	Longest Drought (Years)	Maximum Deficit 10^9m^3	Adjusted Range 10^9m^3	Hurst Coefficient
Historical	88.28	14.60	0.246	11.00	-143.70	416.99	0.820
PAR(1)							
Mean	88.24	15.63	0.520	7.84	-107.64	237.20	0.658
St. Deviation	1.78	1.28	0.276	2.08	29.42	61.46	0.058
PARMA(1,1)							
Mean	88.27	15.04	0.520	7.56	-98.35	217.12	0.646
St. Deviation	1.60	1.20	0.270	2.01	27.14	55.54	0.058
PARMA(2,1)							
Mean	88.26	14.24	0.538	7.40	-88.73	197.22	0.636
St. Deviation	1.43	1.11	0.269	1.88	24.40	49.95	0.058
PARMA(1,1)(1,1)							
Mean	89.04	15.84	0.517	11.11	-167.76	395.93	0.773
St. Deviation	10.25	2.58	0.253	4.67	91.97	159.07	0.070
PARMA(1,1)(3,0)							
Mean	88.15	15.12	0.513	10.45	-150.39	315.18	0.734
St. Deviation	2.71	1.45	0.280	3.19	51.56	93.67	0.060

5. SUMMARY AND CONCLUSIONS

An important aspect in stochastic simulation is to find a model which is capable of reproducing both short term and long term statistical characteristics of the historical data. In this paper, a new periodic stochastic model is presented for generating monthly flows of the Nile River which is capable of preserving annual statistics besides the preservation of monthly statistics. The proposed model, called the multiplicative periodic autoregressive moving average XPARMA(p,q)$(P,Q)_\omega$ model, represents a generalization of the multiplicative ARMA model of Box and Jenkins (1976) and an extension of the traditional PAR(p) and PARMA(p,q) models. The model parameters can be estimated by the method of least squares. The XPARMA model has been applied to simulate the Nile River monthly flows at Aswan and extensive computer experiments have been made to compare the attributes of this model with those of the PAR and PARMA models.

The results of the study have shown that the XPARMA model is a good alternative for modeling and generating monthly flows in rivers of complex variability and dependence structure such as the Nile River flows. The XPARMA models have outperformed the PAR and PARMA models in preserving monthly and annual historical statistics. In addition, low order XPARMA models require the estimation of considerably less number of parameters (60 parameters for the XPARMA

$(1,1)(1,1)_\omega$ model applied to monthly flows) than those needed for commonly used modeling schemes such as the disaggregation model.

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SOUTHEASTERN ANATOLIA PROJECT (GAP) OF TURKEY: WATER RESOURCES DEVELOPMENT WITHIN THE CONTEXT OF INTEGRATED REGIONAL SOCIOECONOMIC DEVELOPMENT

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ABSTRACT

Water resources on a regional scale are mobilized to serve the integrated socioeconomic development of a 75,000 km^2 area in southeast Turkey. The 31 billion-dollar project comprises not only of water resources projects, but also of investments in all development-related sectors such as agriculture, energy, transportation, telecommunications, healthcare, education, urban and rural infrastructure, in an integrated manner. The project is one of the largest of its kind in the world. The water resources development program includes 22 dams, 19 hydropower plants and the irrigation network for 1.7 million hectares of land. This paper, after establishing the general development framework, deals with the water resources development aspect of the project. Approaches, studies, projects and implementations in the water resources planning-to-management spectrum are described from a sustainable-development perspective.

1. PROJECT AREA

GAP project area lies in southeast Turkey, covering eight provinces, corresponding to approximately 10 percent of Turkey's total population and surface area. The project area includes watersheds of the lower Euphrates and Tigris Rivers and the upper Mesopotamian plains. The total surface area is 75,000 km^2, of which 42.2% is cultivated (36% rain-fed farmland), 33.3% pastures, 20.5% forest and bush. Average gradient over 94% of the total surface area is less than 12%, which is generally accepted as the threshold for cultivability. Salinity and alkalinity problems are minimal, and soils with insufficient drainage do not constitute any major proportion. While wind erosion is minor, water erosion of moderate to strong levels are observed to a somewhat larger extent.

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The split of population between urban and rural areas are respectively 56 % and 44%, as compared to 59% and 41% at the national level. The rate of population growth in the project area is 3.4% per annum, a point higher than the national average.

Basic infrastructure in the project area is generally adequate, with 98% of all villages and rural units linked to the road network, and 95% of all villages electrified. Over 70% of rural settlements possess water supply, whereas school enrollment rates and healthcare facility use remain below the respective national averages. Urban centers, with better living standards and services attract rural population in substantial proportions and are therefore threatened by an increasingly inadequate infrastructure.

2. GAP AS A SOCIOECONOMIC DEVELOPMENT PROJECT

Project GAP aims at removing a socioeconomic "gap" between the project area and more developed regions in Turkey. The region's economy is dominated by the agricultural sector and agriculture is largely practiced under rain-fed conditions. The region's contribution to Turkey's gross domestic product (GDP) is a low 4%, and to the national value-added in the industrial sector is an-even lower 2%.

The region, on the other hand, is rich in soil and water resources. The Tigris and Euphrates Rivers together provide about 28% of all national water supply by rivers; the economically irrigable land area in the region amounts to 20% of that for the whole country.

In order to improve the economy of the region, mobilization of the water and soil resources on a regional scale has been planned, within the framework of integrated, multi-sectoral regional development. As such, the 31 billion-dollar project comprises not only of water resources projects, but also of investments in all development-related sectors such as agriculture, energy, transportation, telecommunications, healthcare, education, urban and rural infrastructure, in an integrated manner. The project is one of the largest of its kind in the world. The water resources development program aims at producing 27 billion kilowatt-hours of hydroelectric energy and irrigating 1.7 million hectares of land. A total of 22 dams, including the sixth largest-volume dam in the world (Atatürk Dam), 19 hydropower plants, the Şanlıurfa Irrigation Tunnel System -the largest of its kind, numerous irrigation networks, canal systems constitute the physical groundwork in water resources (Table 1).

Water resources development of this size and scale is bound to have effects and implications that go far beyond irrigation-related activities, touching every facet of life and involving all social and economic sectors. On-farm development, farmer training and extension programs, agricultural input provision, credit and marketing arrangements, agro-processing, related rural infrastructure, operation and maintenance of the extensive irrigation system, environmental protection, preservation of historical and cultural heritage, social attitudes and expectations are some of the issues that need to be addressed in water sector.

Project planning and implementation are done based on a *Master Plan* and on an *Action Plan.* Macro-level planning and management, coordination, monitoring-evaluation and implementation in selected areas are carried out by the *Regional Development Administration.*

A comprehensive investment program is under implementation: Out of the total public sector finance requirement of 31 billion U.S. dollars, 10 billion has already been invested, one billion dollar is being spent this year.

The project will double the nation's hydroelectric production, increase the irrigated areas by 50%, more than quadruple the gross regional product, and more than double the per-capita income in the region.

Some of the larger project components are ready or near completion, such as the Atatürk Dam, which has produced over 7 billion kilowatt-hours of hydroelectric energy since it began production in mid-1992. Irrigation on a large-scale basis will start when 26.5 km-long twin Urfa-tunnel system becomes operational in 1994. A major new University has been established, a transregional highway and railroad line are being designed, and studies for an international airport are underway. Construction work continues in sectors such as agriculture, urban and rural infrastructure, telecommunications, industrial zones, healthcare and education on a regional scale.

3. INTEGRATED REGIONAL DEVELOPMENT

The strategy adopted for the region's development has the following four basic components:

(i) to develop and manage the region's soil and water resources for irrigation, industrial and urban uses in an optimal manner,

(ii) to improve the land use through optimal cropping patterns and agricultural practices,

(iii) to promote agro-industries and those based on indigenous resources, and

(iv) to provide better social services, education and employment opportunities in order to keep local people where they live as well as to attract qualified personnel to the project area.

The GAP Master Plan's basic development scenario is to make the region an agro-related export base.

The project is socially justified as it is intended to significantly improve the living standards of the local people, increase the per-capita income, and provide employment opportunities. It is also economically justified as it will fundamentally alter the economic structure from an agriculture-dominated one to a contemporary one. As a result of large-scale irrigation, agricultural production, productivity, and crop variety will increase substantially. Agricultural production estimates are given in Figures 1a and 1b, for primary and secondary products, respectively. This change, in turn, will create a chain reaction, boosting agro-related industries and services. The expected structural change as a result of project implementation is shown in Figure 2. The expected changes in the gross regional product and value added in economic sectors are shown in Figure 3.

The basic development principles in the Southeastern Anatolia Project are:

(i) Integrated, regional development as opposed to project-specific development and sectoral planning, and

(ii) Sustainable development.

GAP is considered to be one of the biggest projects in the world today. Recently, it has been rated as one of the wonders of the modern world by international media.

4. WATER RESOURCES DEVELOPMENT

The history of efforts to mobilize water for human welfare could provide a relatively comprehensive history of civilization itself, be it irrigation, water power, navigation or flood control. Irrigation is still the major use of water and will continue to be so, in the foreseeable future. Although the principal nature of the issue remains unchanged throughout eras, the situation is much more complex in today's world. The relatively easy or manageable world of water resources development is a thing of past; our natural resources are less abundant; additional uses and a much bigger population compete for the available resources. The challenge to satisfy the need for the basic uses such as drinking, food production, and hygiene has been aggravated by new needs as a result of the sophistication of the society, high urbanization and industrialization. These are no less valid in GAP's water resources development than in any other project. On the other hand, the scale and nature of integrated development in GAP makes it essential, from planning to operation, to properly take into account all elements connected to optimality in resource use and sustainability in development, especially in water resources development.

5. SUSTAINABILITY

Sustainability is not a new concept, its components and underlying ideas have been around for a long time. Once largely limited to academia and intellectia, it has now evolved into a concept that makes wonderfully good environmental sense, but quite good economic, social, and engineering sense, as well.

5.1 Irrigation Sustainability

Sustainability in irrigation is probably one with more engineering inputs, compared to the others addressed in this paper. It involves prerequisites such as the quality of design and construction as well as proper system operation and maintenance, effective monitoring and evaluation and a commitment to improve system performance when the need is detected. Some of the external factors involved in irrigation sustainability are the quality of the upper drainage basins and implications from agricultural, environmental and spatial development policies and practices.

Some of the major problems plaguing irrigation sustainability are water losses (due to inappropriate method or technology, lack of education and training and improper infrastructure), water logging/salinity (due to excessive watering and insufficient drainage), and erosion. A major general problem is a preoccupation with "instant gratification", that is, using the available resource for water supply for quick benefits, ignoring proper infrastructure, skipping drainage, neglecting education and training. Very typical to less-developed countries, money saved from proper drainage and canal lining is spent to extend water supply, thus aggravating the problem. Other reasons are the lack of incentives for water conservation, lack of legislative or institutional arrangements, enforcement difficulties and inadequate education. We now know that, regardless of the reason, this leads to problems that are eventually a lot more expensive to solve.

5.2 Sustainability: Agricultural System and Components

Influenced by the irrigation system performance, sustainability in producing crops that produce value added is also affected by a number of more complex and broader issues such as:

(i) markets: existence, prices and accessibility,

(ii) economics: ability to produce value added,

(iii) production system: soil quality, land-use patterns and competing alternatives, cropping intensity, and

(iv) engineering and supply system: proper infrastructure, knowledge and services (agricultural research and development, proper provision and use of agri-inputs and credit, extension and training, physical infrastructure-road networks, communication facilities, storage and transport.

5.3 Environmental Sustainability

A better understanding of the rapid deterioration of natural resources in the world and the dramatic (negative) consequences of massive development in 1950's have made the preservation and enhancement of the water-soil-air ecosystem a very high priority consideration in development.

Two important implications of environmental sustainability are water quality degradation and irrigation related diseases (crop, animal, human).

Other development-related environmental issues include air, water and noise pollution due to industrial development, urbanization, and inadequate infrastructure. Creation of large reservoir lakes and extensive irrigation schemes calls for special attention.

5.4 Societal Sustainability

The beneficiaries and users in irrigation development are people. Sustainability in this sense would require that the needs, wants, and "don't wants" of people be incorporated in the whole spectrum -from planning to operation- in an appropriate manner. The main issue here is, in broad terms, to properly understand the

- impact of development on social attitudes, and

- impact of societal expectations and needs on development

and act accordingly. Development should not bring about such easy compromises as implementing "primitive" systems to escape potential resistance or imposing values on people.

Societal sustainability concept is typically taken for granted in western countries, and is yet to be recognized in many developing countries. Nevertheless, earlier with the progress in telecommunications and information technology and now with the large-scale democratization in the world, this is bound to gain greater importance in development in near future.

5.5 Concluding Remarks

Sustainability has definitely imposed a new condition for all facets of development. It has also made it essential to adopt an integrated approach to development, as opposed to the fragmented, project-specific (or sector-specific at best) approach of the past. An immediate implication of this has been the need for interdisciplinary collaboration and for a thorough understanding of the system dynamics. Regardless of the issue or the individual situation at hand, we have to deal with or take into account a new set of concepts including environmental considerations, institutional arrangements, legislation, user participation, interagency cooperation, education and training, research and development, use of technology not only in the design or in physical structures and equipment, but also in operation, management, monitoring, and data acquisition.

We know that sustainable water resources development can be achieved. Mistakes made in the past, in the developing and the developed world alike, provide us valuable information as to the possible consequences. To re-cite some of these mistakes:

- fragmented or project-specific approach as opposed to integrated development,
- lack of concern, legislation, institution or enforcement for water-related issues,
- insufficient or misallocated funds,
- improper, inadequate or non-existent physical infrastructure,
- lack of proper drainage,
- upper watershed degradation,
- improper maintenance, operation and management practices,
- non-use or mis-use of technology,
- lack of user participation,
- neglecting societal needs and expectations.

6. DEVELOPMENT APPROACH IN SOUTHEASTERN ANATOLIA PROJECT

GAP is planned and implemented as an integrated multi-sectoral regional development project in which water and soil resources provide the basis for development. The project covers the development-related social and economic sectors such as agriculture, forestry, industry, transportation, health care, education and tourism together with interactions among them.

In developing the region's soil and water resources integrated planning, optimality and sustainability concepts are taken into account in the respective plans and projects as well as through a number of studies/principles:

- Canals and flumes are always lined,

- Drainage is always provided,

- Use of urban wastewater after treatment in irrigation and watering is under consideration,

- Modern technology is adopted where appropriate: pressurized irrigation, modern canal regulation with downstream control and remote supervision,

- Major model development project for management, operation and maintenance of irrigation systems under way: institutional issues, water charges, farmer participation aspects included,

- On-farm development, leveling and land consolidation to improve economic viability and to prevent salinity and water logging,

- A large-scale agrarian reform implementation is under way,

- Further water savings are aimed when appropriate such as conjunctive use, and re-use of irrigation return water,

- Major watershed rehabilitation in upper basins (World Bank funding),

- Farmer training and extension programs on large-scale basis (World Bank funding),

- Major agricultural research and development project in implementation,

- Optimal crop pattern and agricultural marketing project completed, taking into account, domestic, regional and international markets,

- Mechanization, input, storage and credit requirements are estimated and provision planned,

- All related physical infrastructure needs are estimated and construction programmed,

- Impacts on crop, animal and human health studied (GAP Health Sector Master Plan),

- Formal and vocational educational needs to serve development objectives and requirements estimated, implementation program prepared (GAP Human Resources, Education and Training Master Plan),

- Environmental effects taken into account, environmental impact assessment is under way,

- Watersheds are protected through legislation, development (especially urban and industrial) in drainage basins closely controlled,

- Societal attitudes and expectations are taken into account, extensive research and field work by sociologists conducted,

- Governments completely committed to proper completion of GAP, financing provided, institutional and legislative arrangements are made, e.g. GAP Administration has been established to plan, coordinate, monitor project activities and to control spatial development; monitoring/evaluation is followed by measures,

- Technology and modeling capabilities are vastly utilized, e.g.
  - data acquisition and monitoring,
  - use of geographical information systems in appropriate areas,
  - computer-aided manufacturing in Atatürk Dam,

- roller-compacted concrete technology in Atatürk Dam's cofferdam,
- laser-guided irrigation tunnel construction,
- hydrological and reservoir operation modeling,
- global circulation model under way to improve forecasting capabilities.

7. CONCLUSION

Turkey is implementing an integrated development project based on water resources. The project will greatly enhance the living standards and income levels of the people living in a 75,000 km^2 area that corresponds to 10% of the national population and surface area. Project encompasses all sectors in an integrated and coordinated manner.

The project enjoys wide-scale political and popular support. An administration has been established specifically for project-related activities.

Water resources development takes into account sustainability requirements; engineering, environmental, social, institutional, legislative considerations are incorporated, modern technology is utilized.

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Opinions presented in this paper are those of the author and do not necessarily reflect those of the Turkish Government.

Table 1. Water Resources Projects in GAP

G A P

Euphrates Basin	GAP	Tigris Basin
5304 MW	Capacity : 7 476 MW	2172 MW
20098 GWh	Production : 27 345 GWh	7247 GWh
1091203 ha	Irrigation Area : 1 693 027 ha	601824 ha
	Number of Dams : 22	
	Number of HEPP : 19	

Project	Capacity (MW)	Production (GWh)	Irrigation Area (Ha)	Present Stage
I. Karakaya Project	1800	7354		
Karakaya Dam & HEPP	1800	7354		OP
II. Lower Euphrates Projects	2450	9024	706281	
Atatürk Dam & HEPP	2450	8900		U/C-OP
Şanlıurfa HEPP	50	124		U/C-OP
Şanlıurfa Irr.Tunnel				
a) Şanlıurfa Tunnel & Irrigation			141835	U/C-OP
b) Mardin-Ceylanpınar Gravity Irrigation			185639	M/P
c) Mardin-Ceylanpınar Pumped Irrigation			149000	M/P
Siverek-Hivan Pumped Irrigation			160105	Rec
Bozova Pumped Irr.			69702	Rec
III. Border Euphrates Prj.	852	3168		
Birecik Dam & HEPP	672	2516		U/C
Karkamış Dam & HEPP	180	652		D/D
IV. Suruç-Baziki Project			146500	
Suruç Baziki Plain Irr.			146500	Rec
V. Adıyaman-Kahta Project	195	509	77824	
Çamgazi Dam & Irrigation			6536	U/C
Gömikan Dam & HEPP			7762	M/P
Koçali Dam & HEPP	40	120	21605	M/P
Sırımtaş Dam & HEPP	28	87		M/P
Fatopaşa HEPP	22	47		M/P
Büyükçay Dam, HEPP & Irrigation	30	84	12322	M/P
Kahta Dam & HEPP	75	171		M/P
Pumped Irr. from Atatürk Reservoir		-	29599	M/P
VI. Adıyaman-Göksu	7	43	71598	
Çataltepe Dam Irr.			71598	F/S
Erkenek HEPP	7	43		F/S
VII. Gaziantep Project			89000	
Hancağız Dam & Irr.			7330	OP
Kayacık Dam & Irr.			13680	U/C
Kemlin Dam & Irr.			1969	F/C
Pumped Irr. from Birecik Reservoir			66021	F/S
INDIVIDUAL PROJECTS	14,4	42	35440	
Nusaybin Irrigaton			7500	OP
Çağçağ HES	14,4	42		OP
Akçakale Grounwater Irr.			15000	OP
Ceylanpınar			9000	OP
Hacıhıdır Project			[illegible]	[illegible]

Project	Capacity (MW)	Production (GWh)	Irrigation Area (Ha)	Present Stage
VIII. Dicle-Kralkızı Porject	204	444	126080	
Kralkızı Dam & HEPP	94	146		U/C
Dicle Dam & HEP	110	298		U/C
Dicle Right Bank Gravity Irrigation			52033	U/C
Dicle Right Bank Pumped Irrigation			74047	U/C
IX. Batman Project	198	483	37744	U/C
Batman Dam & HEPP	198	483		U/C
Batman Left Bank Gravity Irrigation			9574	U/C
Batman Left Bank Pumped Irrigation			9412	F/S
Batman Right Bank Gravity Irrigation			18758	D/D
X. Batman-Silvan Project	240	964	257000	
Silvan Dam & HEPP	150	623		Rec
Kayser Dam & HEPP	90	341		Rec
Dicle Left Bank Gravity Irrigation			200000	Rec
Dicle Left Bank Pumped Irrigation			57000	Rec
XI. Garzan Project	90	315	60000	
Garzan Dam & HEPP	90	315		Rec
Garzan Irrigation			60000	Rec
XII. Ilısu Project	1200	3833		
Ilısu Dam & HEPP	1200	3830		D/D
XIII. Cizre Project	240	1208	121000	
Cizre Dam & HEPP	240	1208		D/D
Nusaybin-Cizre Irr.			89000	Rec
Silopi Plain Irr.			32000	Rec
INDIVIDUAL PROJECTS			25562	
Devegeçidi Project			7500	OP
Silvan I & II Irrigation			8040	OP
Nerdüş Irrigation			2740	OP
Çınar-Göksu Project			3582	U/C
Garzan-Kozluk Irrigation			3700	U/C

Note : Individual project are not included in grand total

Legend : OP- In operation
U/C- Under construction
D/D- Detailed design completed
F/S- Feasibility study
M/P- [illegible]

Figure 1a. Agricultural Production in GAP (Primary Products)

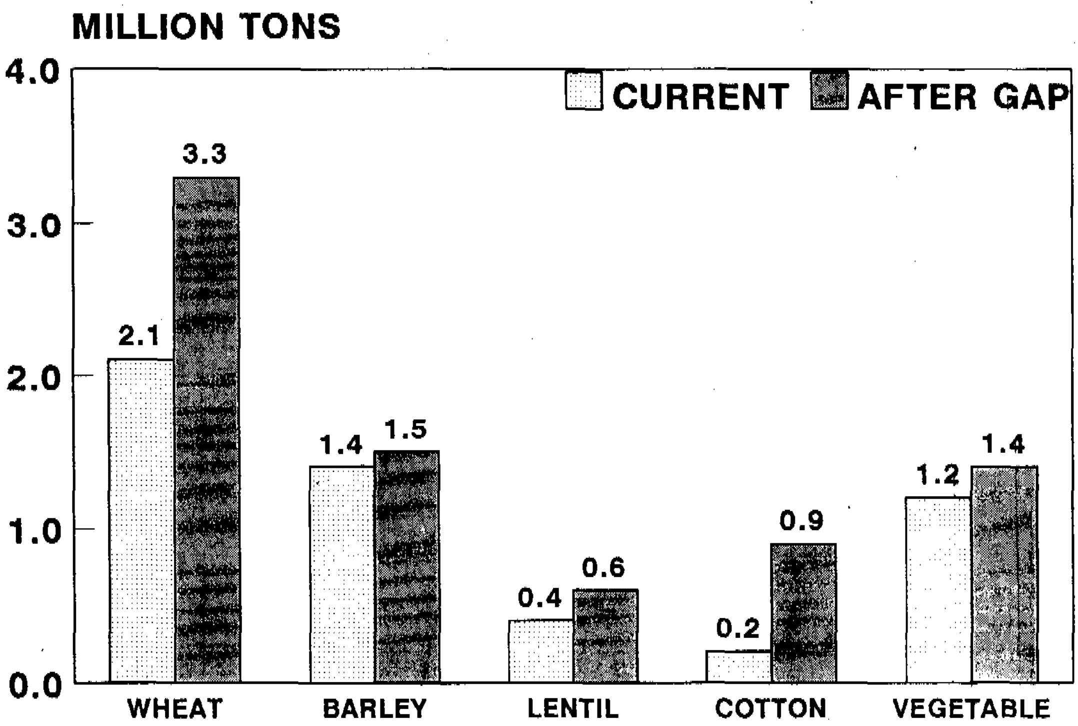


Figure 1b. Agricultural Production in GAP (Secondary Products)

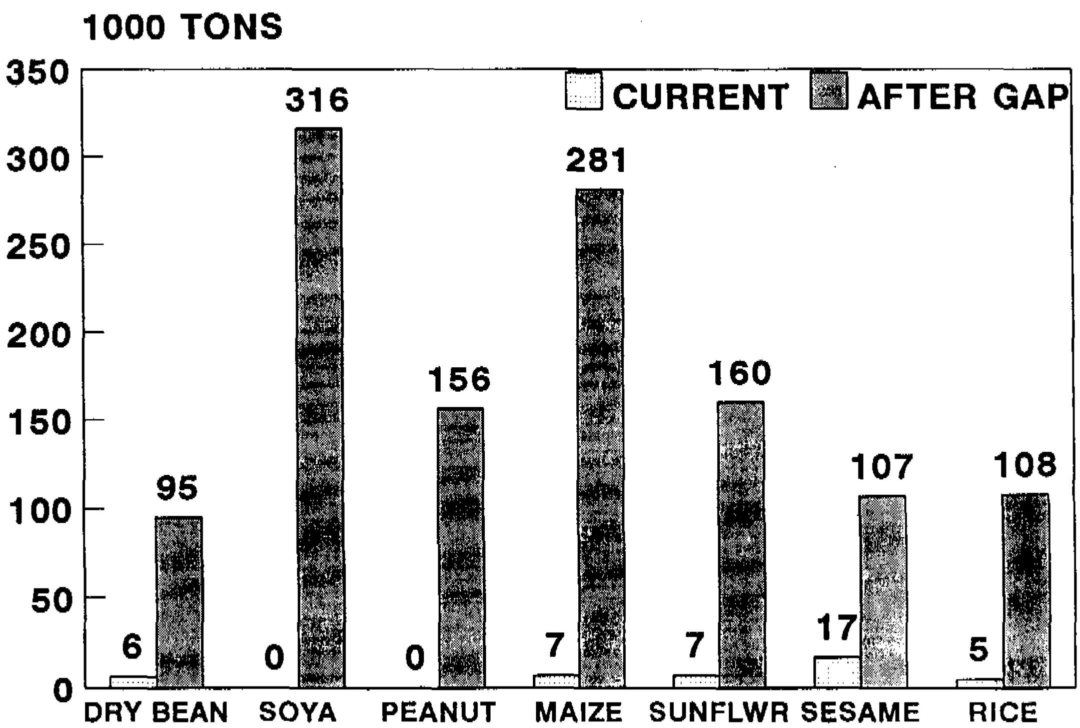
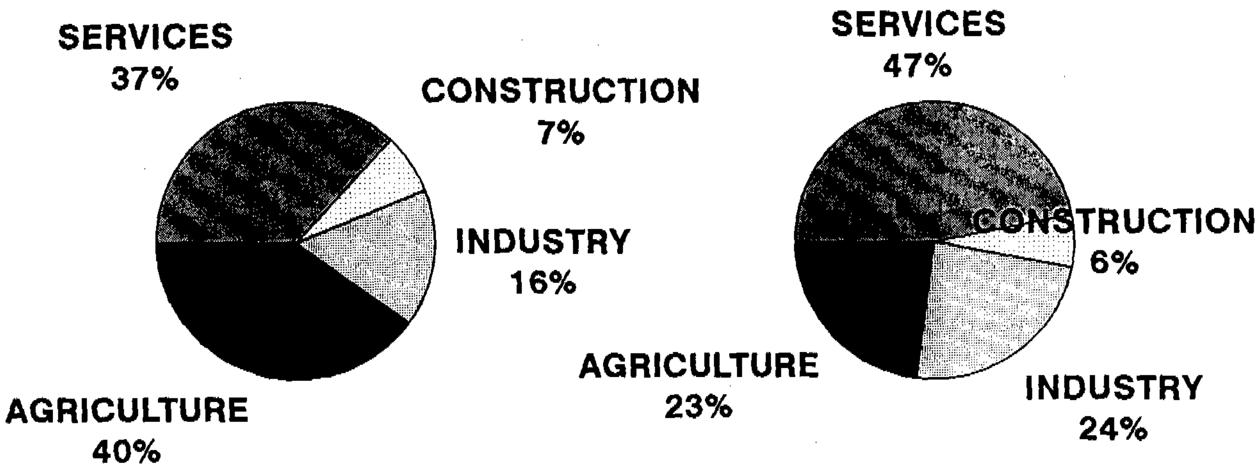


Figure 2. Economic Structure Before And After GAP

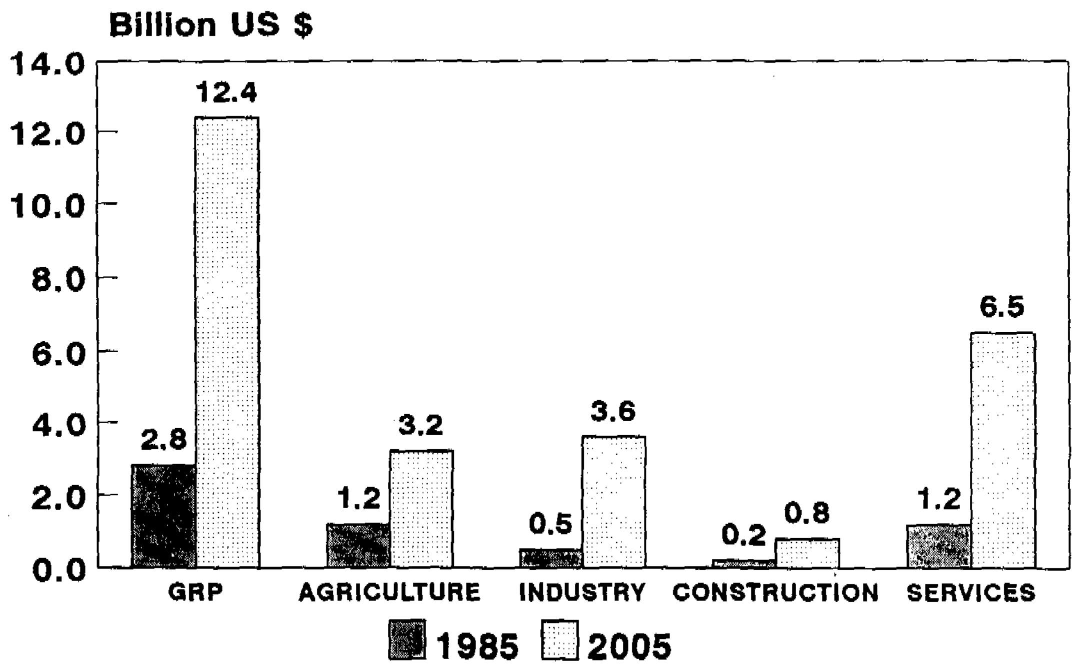


1985
G.R.P. Index : 100

2005
G.R.P. Index : 445

Share of sectoral value added in GRP

Figure 3. Value Added In Economic Sectors Before And After GAP



In 1993 prices

WATER RESOURCES AND WATER DEMAND ASSESSMENT IN THE SALAWIN RIVER BASIN IN THAILAND

Pushpa Raj Onta[1], Rainer Loof[1], Ashim Das Gupta[1], and Ricardo Harboe[1]

Abstract: This paper deals with an assessment of water resources and water demands in the Salawin river basin in Thailand within the context of national level water resources planning. The study area covers about 17,920 sq.km.. Rainfall varies widely in the basin, with a representative mean annual value of about 1180 mm. The average specific discharge ranges from 6.13 to 41.88 l/sec/sq.km. and the total annual inflow amounts to 8565.67 million cubic meters (MCM). Ground water potential is limited. The qualities of both surface water and ground water are mostly found to be good. Irrigation is the major user of water. Annual irrigation water use in 1993 and 2006 is estimated to be 616.93 MCM and 1223.38 MCM respectively. Gross domestic water demands in 1993 and 2006 are 11.78 MCM and 14.29 MCM. Similarly, the water demands for industry, tourism and livestock are 1.98 MCM, 0.52 MCM and 1.95 MCM respectively in 1993, and 21.18 MCM, 0.65 MCM and 2.45 MCM respectively in 2006. Water demands for other purposes are likely to be minimal. With a favorable water supply and demand situation in the study area, the proposals to divert water from the Salawin river basin to the adjoining basins, where there are acute water shortage problems, appear to be reasonable.

Ce papier traite avec l' évaluation des ressources et des demandes en eau dans le bassin de la rivière Salawin en Thailande, en vue de la contexte de la planification des ressources en eau au niveau national. La région d' étude couvre environ 17920 Km^2 . La préciptiation varie largement dans le bassin avec une valeur moyenne annuelle représentative de 1180 mm environ. Le débit moyen spécifique va de 6,13 ā 41,88 $l/s/Km^2$ et l' afflux annuel total s'élève ā 8565,67 millions m^3 (MCM). Le potentiel des eaux souterraines est limité. Les qualités des eaux superficielles et souterraines sont découvertes principalement en bon état. L' irrigation est l'usage principale des eaux. L' estimation d' utilisation annuelle des eaux pour l' irrigation en 1993 et 2006 est 616,93 MCM et 1223,38 MCM respectivement. Les demandes brutes intérieures en eau en 1993 et 2006 sont 11,78 MCM et 14,29 MCM. De la même façon, les demandes en eau pour l' industrie, la tourisme et l' élevage sont 1,98 MCM, 0,52 MCM et 1,95 MCM en 1993 et 21,18 MCM, 0,65 MCM, et 2,45 MCM en 2006, respectivement. Les demandes en eau pour les autres buts sont probablement minimales Avec l' alimentation favorable en eau et la situation des demandes dans la région d' étude, les propositions pour dévier des eaux du bassin de la rivière Salawin au bassins voisins oū il y a des problème aiguës du manque d'eau semblent raisonables.

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1. INTRODUCTION

The Salawin river basin is one of the 25 major river basin studies currently being considered under the water resources programme of the National Economic and Social Development Board (NESDB) in Thailand. The purpose of these studies is to provide an overview of the existing water resources situation at the basin level and to determine the potential for further water resources development. These studies are being coordinated by the three main agencies responsible for various aspects of water resources development in Thailand, namely the Royal Irrigation Department (RID), the Electricity Generating Authority of Thailand (EGAT) and the Department of Energy Development and Promotion (DEDP) or formerly National Energy Administration (NEA). There are several other agencies involved in the water sector at various levels in Thailand which are also contacted during the course of this study.

This paper deals with the assessment of water resources and water demands in the Salawin river basin within the context of national level water resources planning. Due to lack of space, only the summary information and results are provided. For details, reference may be made to AIT(1994).

2. PHYSICAL FEATURES

The Salawin river basin is located in the northwestern part of Thailand, approximately between latitude 16° 15′ - 19° 45′ north and longitude 97° 20′ - 99° 0′ east (Figure 1). The study area covers 17,920 sq.km. which is about 3.5 percent of the total national land area and which includes the whole of Mae Hong Son province and some parts of the provinces of Tak and Chiang Mai. The Salawin river itself is an international river originating from Tibet and runs approximately 2,200 km in a north-south direction through Southern China, Myanmar, and Thailand before draining into the Andaman Sea. The study area, which is the portion of the river basin including Mae Nam Moei lying inside Thailand, thus covers about 60 percent of the total drainage area of the Salawin river basin (i.e. 29,500 sq.km) at Ban Mae Pua in Thai-Myanmar border.

The study area is characterized by steeply dissected mountainous topography, relatively fragile watershed and is covered by many types of forests. The present land use consists of forests and range land (approx. 90%), agricultural land (9%) and others such as built-up areas, etc. (1%). As in other parts of Thailand, agricultural land has increased at the expense of forests over the last couple of years. Most of the agricultural land is observed near Mae Sot in the Tak province. The land system suitable for paddy is quite limited. Mostly rainfed paddy and upland crops prevail; however, some favorable areas do have supplementary irrigation facilities. About 42.0% and 44.7% of the total agricultural land in 1991/1992 were occupied by paddy and upland crops respectively. Tree crops, grass land and others occupied the rest.

The study area consists of consolidated rocks of various types ranging from Tertiary to Carboniferous, mainly Paleozoic and Mesozoic sedimentary rocks and Mesozoic granite. The soil conditions fall into the following groups: (a) alluvial soils on recent alluvial plains, which occupy a small area as narrow strips along rivers and streams, with mostly young and fertile loamy soils mainly used for rice cultivation or upland and tree crops in places; (b) undulating to rolling alluvial

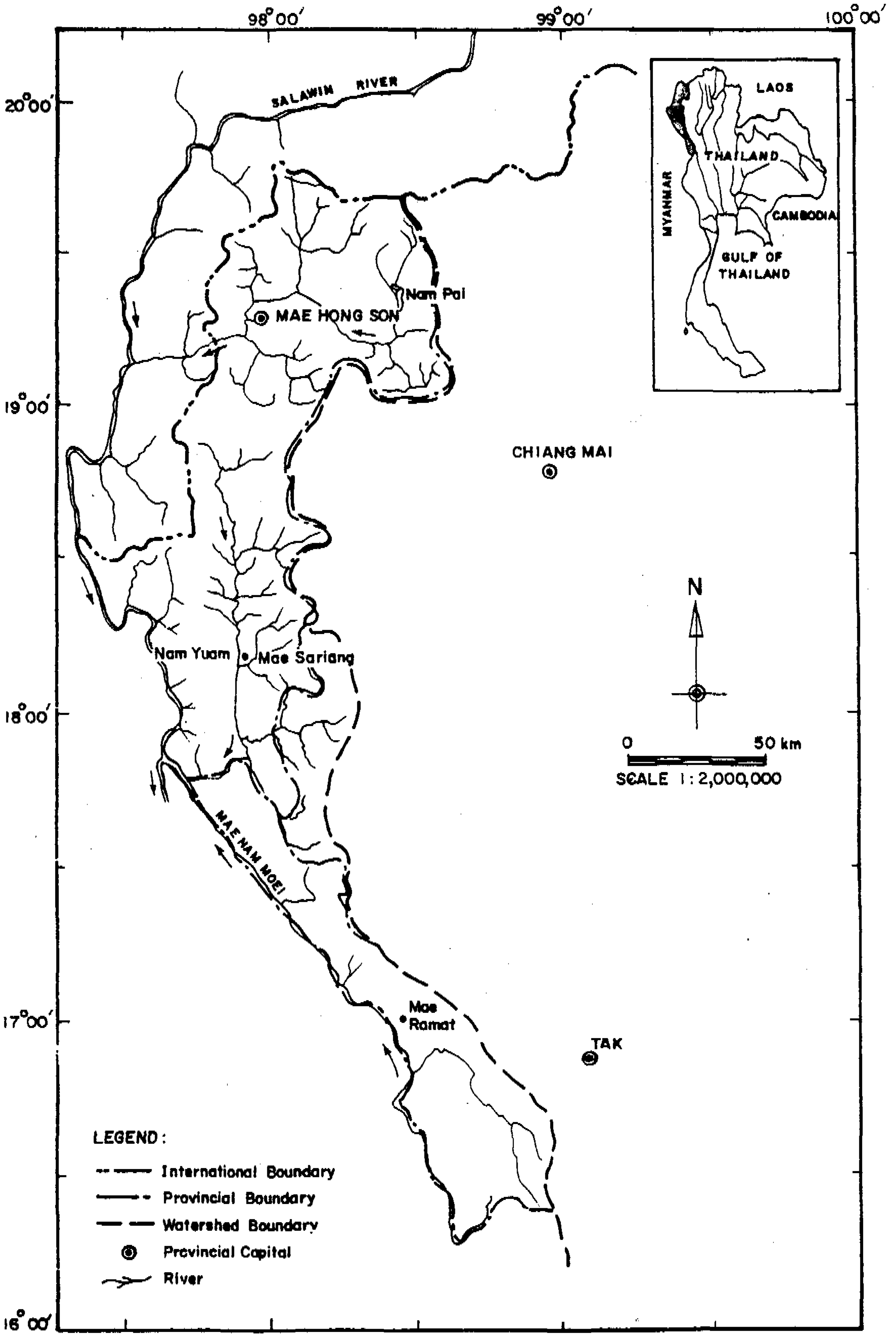

Figure I Location Map of Salawin River Basin terraces which occupy high position between recent alluvial plains and mountainous areas consist mainly of red yellow podzolic and reddish brown lateritic soils. These soils are generally unsuited for field crops; and, (c) soils on undulating to rolling footslopes/hills and steep mountainous area which are not suited for agricultural purposes.

The forestry land classification map of the study area as outlined by the Royal Forestry Department indicates that the 'conservation zone' covers most of the area while 'economic zone', mainly concentrated in Mae Sot area, comes second. 'Agricultural zone' is negligible while other areas (such as residential, etc.) are also quite less. The watershed classification systems and maps for the Salawin river basin are currently being finalized by the Government. It is estimated that Watershed Class 1 (WSC 1 - Protection or conservation forest and headwater source) covers around 70% (mostly WSC 1A - wholly undisturbed, WSC 1B being less than 10%), while the rest is divided almost equally between WSC 2 (Commercial forest), and WSC 3 (Fruit tree plantation), WSC 4 (Upland farming) and WSC 5 (Lowland farming) combined. Thus, the forest and watershed conditions represent constraints in water resources development in the study area.

3. HYDROLOGIC SYSTEM

3.1 General Climate: The general climate of the area is characterized by winter (November-February), summer (February-mid May) and rainy (mid May until October) seasons, influenced by northeast and southwest monsoons. The hydrometeorological data in the study area are recorded by the Meteorological Department (MD), in addition to RID, EGAT and DEDP. All the stations operated by MD in the region are index or synoptic stations recording most of the meteorological variables in addition to rainfall. Altogether, 52 rainfall stations in and around the study area were used for the analysis. Only 3 index stations fall inside the study area but their locations at Mae Hong Son, Mae Sariang and Mae Sot are fairly well distributed.

Mean annual rainfall isohyetal map of the study area is shown in Figure 2. The distribution of isohyets shows that rainfall in the Nam Moei sub-basin, particularly in its lower part, is higher than in the Nam Pai and Nam Yuam sub-basins. It is observed that there is a very wide variation in the annual rainfall values in the study area, ranging from about 900 mm in Om Koi (st. 01007162) to about 3,242 mm in Tha Song Yang (st. 04011801). Mean annual rainfall at the representative station at Mae Sariang is about 1180 mm. Mean monthly rainfall distributions for typical stations are shown Figure 3. It is generally observed that of the total annual rainfall, more than 90% occurs during the six-month period from May to October and more than 2/3rd amount fall during the period June-September (Figure 4). July and August are normally the months of heaviest rainfall. The variation of monthly rainfall is much higher than that of annual values.

Although rainfall distribution in the study area was found to vary considerably, other meteorological variables were not. The data from Mae Sariang station (st. 03048325) may, therefore, be considered representative of the whole study area. The mean monthly temperature is quite uniform throughout the year ranging from about 21 to 30°c. Extreme maximum temperature of 43°c is observed in April whereas extreme minimum temperature of 3.3°c is observed in December. The annual mean, mean maximum and mean minimum values of relative humidity are 76%, 94%

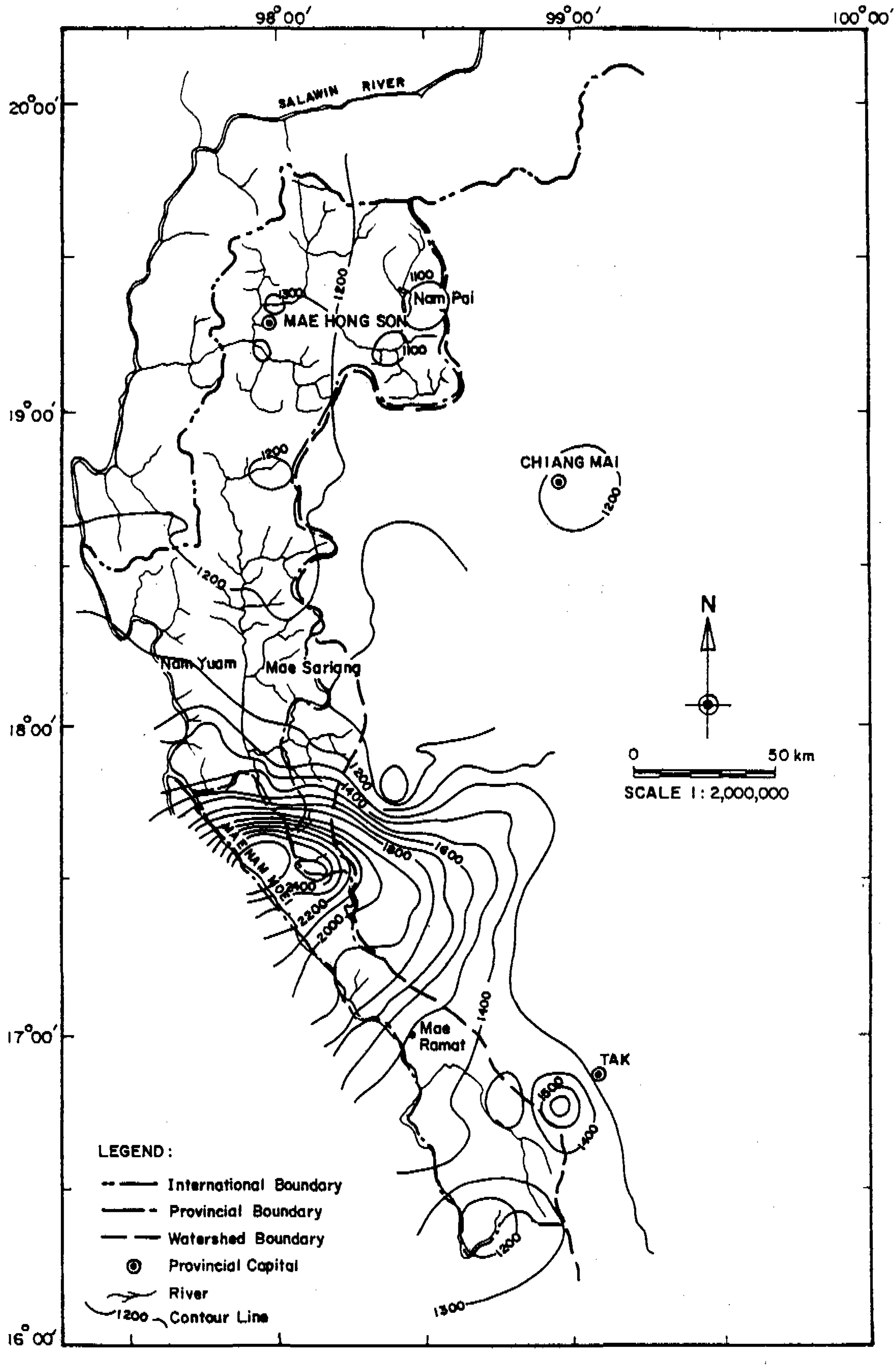

Figure 2 Isohyetal Map of Mean Annual Rainfall (mm)

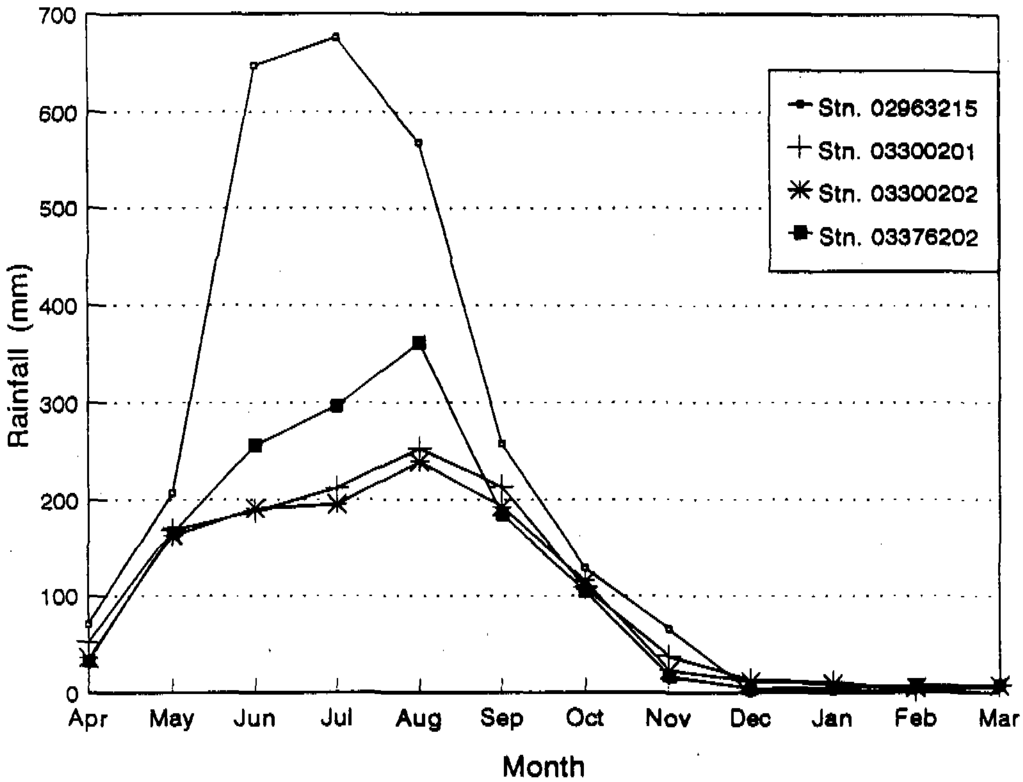

Figure 3 Distribution of Mean Monthly Rainfall for Typical Stations

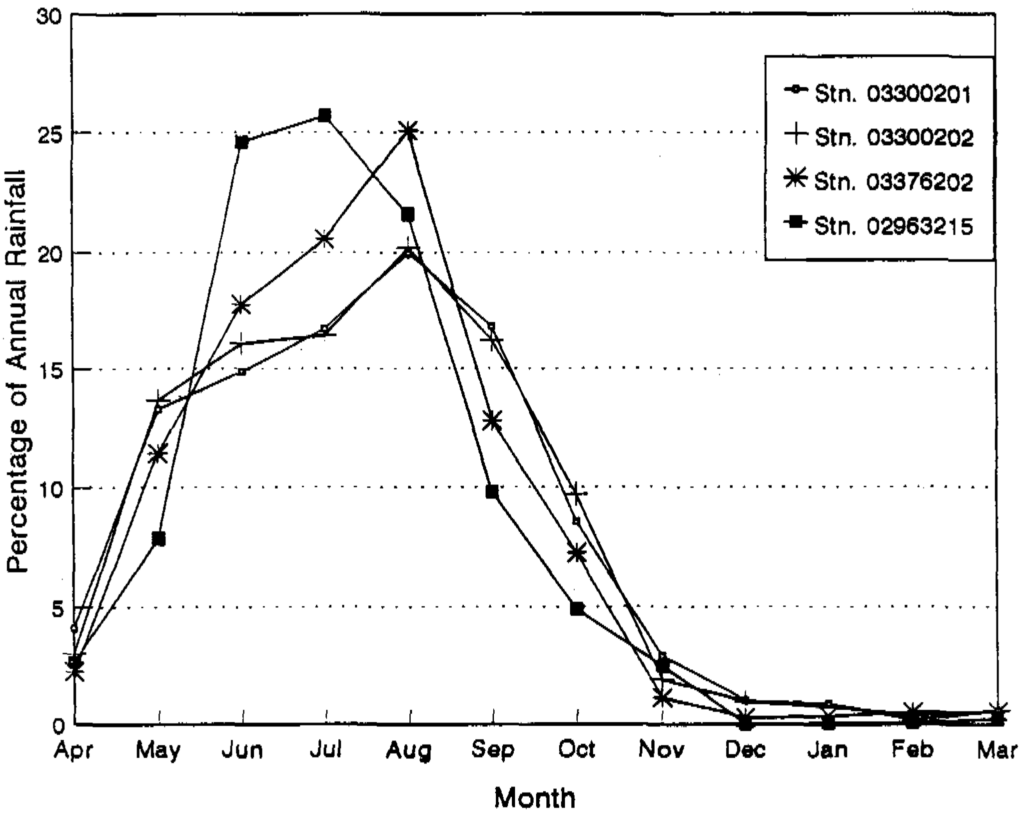

Figure 4 Percentage Distribution of Mean Monthly Rainfall for Typical Stations

and 53% respectively.

The mean monthly pan evaporation values at the Mae Sariang index station shows that the maximum value of 192.7 mm occurs in April and the minimum value of 84.9 mm occurs in December. Generally speaking, the observed pan evaporation values are high when the temperature is high (March-May) and low when the relative humidity is high (June-December).. The mean annual evaporation at the station is 1413.0 mm while the mean annual rainfall is only 1180.3 mm. These evaporation values are also consistent with the potential or reference evapotranspiration (ETo) values estimated based on the FAO-adapted Modified Penman Method. Annual ETo was computed to be 1474.0 mm (maximum: 198.0 mm in April; minimum: 86.8 mm in December). The net rainfall that is potentially available for runoff occurs only during the period from May to October.

3.2 Surface Water: The complex and mountainous topography of the Salawin river basin has given rise to a similarly complex network of river system. The study area in the Salawin basin may be divided into 18 different sub-basins as shown in Figure 5. For hydrological analysis, the study area may be conveniently divided into three main sub-basins, namely Nam Mae Pai, Nam Mae Yuam and Mae Nam Moei sub-basins (see Figure 5). Three sub-basins, namely, Nam Mae Surin (No. 7), Nam Mae Ngae (No. 15), and the upper part of Mae Nam Salawin (No. 14), have to be separately considered as they are independent.

The list of the stream gauging stations existing in the study area is given in Table 1. The list includes the code of each station. Most of the stations in the area are operated by DEDP and only few are operated by RID and EGAT. The table also shows which of these stream gauging stations cover sediment volume and extreme flow records. There are altogether 35 stream gauging stations. Out of these 35 stations, 14, 12 and 8 lie in the Nam Mae Pai, Nam Mae Yuam and Mae Nam Moei sub-basins respectively, and 1 lies in the main Salawin river. Sediment volume records are available in 23 stations whereas annual extreme value records of flood flows and low flows are available in 24 stations.

For planning purpose, monthly streamflow records should have at least 25-30 years data and should cover the latest possible data. Since many stations had data problems, the HEC4 - Monthly Streamflow Simulation Model developed by the Hydrologic Engineering Center, US Army Corps of Engineers, was adopted to fill up the missing values and reconstitute the flows at all the sites for the period 1965-1991. This model can be used even when the historical sequences of streamflow at various sites in a basin are of unequal length, span over different periods of time or when they are discontinuous over time. The model has the capability of filling in missing data in the historical sequences by taking into account the serial correlation of flows at each station and cross correlation with all other stations in the basin (HEC, 1971).

For HEC-4 application, only 31 stations were considered. Four stations were dropped because they had the flow records of only 2-4 years. Thus, only the stations with more than 7 years of data were considered in the reconstitution of flow data. The total number of 31-stations data were categorized main sub-basinwise into 5 classes (2 each for Pai and Yuam, and 1 for Moei) for the purpose of analysis. It was observed that the model results were satisfactory and consistent

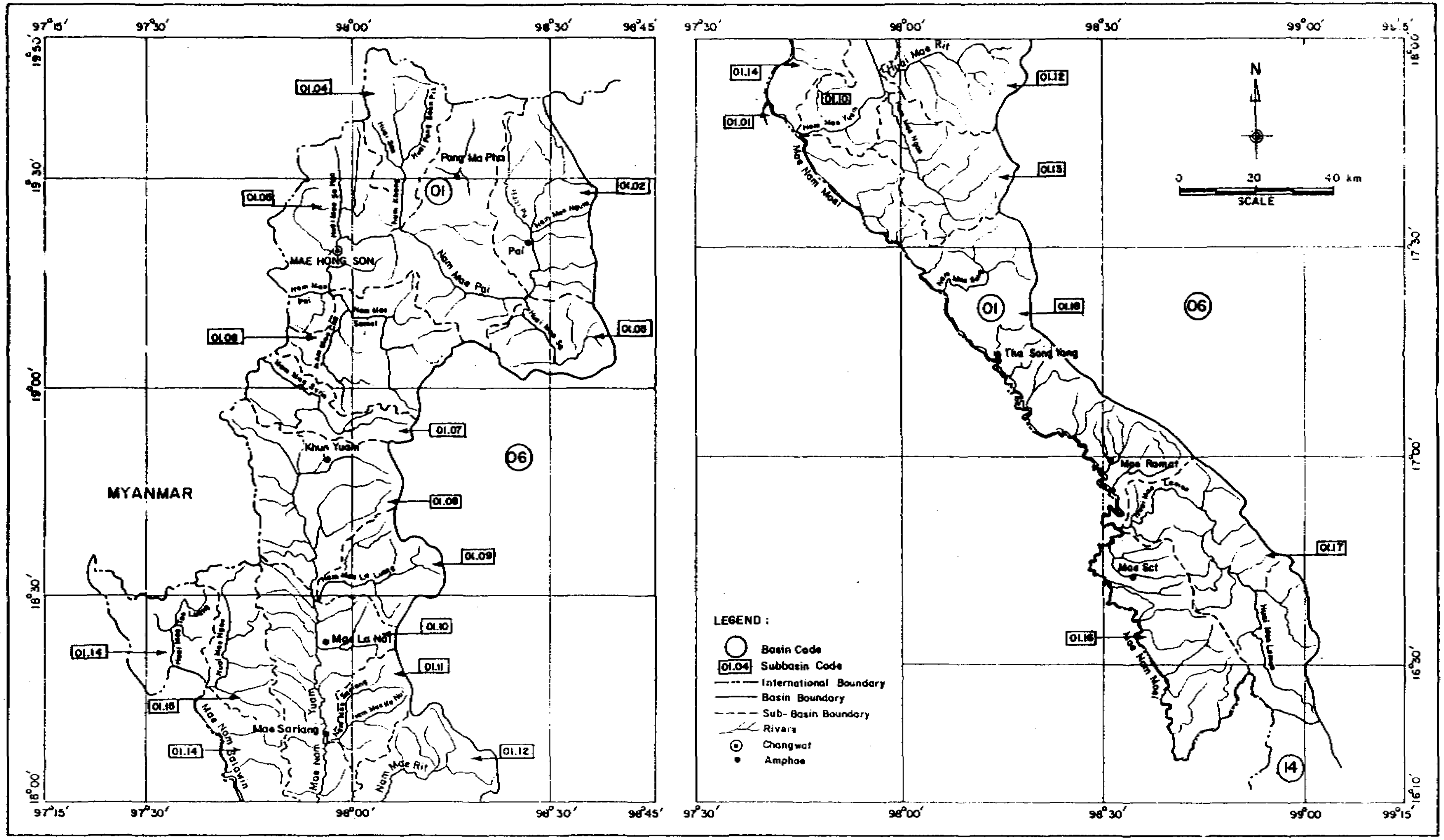

Figure 5 Sub-basins and Stream Network in the Study Area

Table 1 : List of Stream Flow Gauging Stations

No.	Code	River	Station	Latitude	longitude	Drainage Area(Sq.Km)	Remark
1	04011401	Salawin	Ban Mae Mae Pua	17°-58'-48"N	97°-44'-24"E	29500.0	+ x
Nam Mae Pai Sub-basin							
2	04010202	Nam Mae Khong	Ban Mae Na	19°-25'-54"N	98°-24'-06"E	172.0	+ x
3	04010203	Nam Mae Khong	Sop Mae pai	19°-23'-18"N	98°-26'-30"E	260.0	+ x
4	04010201	Nam Pai	Ban Na Chalong	19°-24'-00"N	98°-27'-12"E	369.0	+ x
5	04010204	Huai Mae Ping	Huai Kaeo	19°-17'-18"N	98°-29'-56"E	54.4	+ x
6	04010205	Huai Mae Ya	Sop Huai Mae Ya	19°-14'-24"N	98°-28'-54"E	84.8	+ x
7	04010504	Nam Pai	Dam Site	19°-13'-18"N	98°-20'-18"E	1760.0	+ x
8	04010401	Nam Khong	Ban Mae Suya	19°-32'-06"N	98°-07'-00"E	414.0	* + x
9	04010505	Nam Mae Sa-Nga	Ban Yang Top Sok	19°-28'-30"N	97°-58'-18"E	123.0	+ x
10	04010501	Nam Pai	Pang Mu	19°-21'-30"N	97°-57'-54"E	3770.0	+ x
11	0200SWN7	Nam Mae Hong Son	Mae Hong Son	19°-16'-54"N	98°-00'-54"E	43.6	x
12	0100SW5A	Nam Mae Pai	Ban Tha Pong Daeng	19°-16'-10"N	97°-56'-55"E	4466.0	
13	04010601	Nam mae Samart	Pha Bong	19°-10'-12"N	98°-00'-00"E	589.0	x
14	04010602	Nam Mae Cha	Pha Bong	19°-10'-00"N	97°-58'-12"E	297.0	+ x
15	04010503	Nam Pai	Sop Mae Samart	19°-14'-00"N	97°-56'-00"E	5530.0	+ x
Nam Mae Yuam Sub-basin							
16	04010902	Nam Mae La Luang	Ban Muang Rae	18°-32'-36"N	98°-02'-12"E	268.0	+ x
17	04010901	Nam Mae La Luang	Mae La Luang	18°-32'-12"N	97°-57'-12"E	328.0	+ x
18	04010903	Nam Mae La Luang	Ban Hua La	18°-32'-46"N	97°-57'-34"E	427.0	+ x
19	04011001	Nam Mae Yuam	Sop Han	18°-12'-12"N	97°-56'-06"E	2496.0	+ x
20	04011103	Nam Mae Sariang	Son Mae Om Long	18°-11'-36"N	97°-59'-06"E	229.0	+ x
21	04011101	Nam Mae Sariang	Chom Chaeng	18°-09'-48"N	97°-58'-00"E	378.0	x
22	04011102	Nam Mae Ka Nai	Ban Pha Chi	18°-08'-12"N	98°-00'-42"E	33.5	x
23	01000SW9	Nam Mae Sariang	Ban Mae Sariang	18°-09'-45"N	97°-57'-20"E	375.0	*
24	01000SW2	Nam Yuam	Ban Tha Khan	18°-21'-56"N	97°-56'-06"E	2617.0	*
25	0200SWE1	Nam Mae Rit	Ban Mae Suat	17°-53'-30"N	97°-57'-48"E	1376.0	+
26	0200SWE2	Nam Mae Ngao	Ban Mae Ngao	17°-51'-18"N	97°-58'-12"E	935.0	+
27	04011002	Nam Mae Yuam	Ban Tha Rua Pha Lae	17°-50'-00"N	97°-54'-48"E	4890.0	+ x
Mae Nam Moei Sub-basin							
28	01000SW6	Huai Mae Lamao	Ban Mae Lamao	16°-45'-44"N	98°-45'-14"E	1038.0	
29	0100SW4A	Huai Mae Lamao	Ban Mae Lamao	16°-48'-14"N	98°-44'-47"E	1060.0	*
30	04010701	Nam Mae Lamao	Ban Mae Lamao	16°-48'-36"N	98°-45'-36"E	1100.0	+
31	01000SW1	Huai Mae Lamao	Ban Ko Ko	16°-54'-56"N	98°-37'-57"E	1426.0	
32	04011802	Nam Moei	Mae Ramat	16°-58'-24"N	98°-28'-42"E	6070.0	+ x
33	04011804	Nam Mae U-su	Ban Pho No Tha Tai	17°-20'-48"N	98°-12'-30"E	68.0	
34	04011803	Nam Mae Tan	Ban Dae Pha Tho Tha	17°-15'-48"N	98°-14'-24"E	55.1	
35	04011801	Nam Moei	Tha Song Yang	17°-34'-06"N	97°-54'-54"E	8360.0	+ x

Note: * - Stations having 2-4 years data and ignored in the analysis.
+ - Stations having Sediment Data.
x - Stations having Low flow and Flood Flood Data.

with the correlation coefficients of raw data. Performance of HEC-4 model was found to be very good in that the model preserved the basic statistics of the original data series very well. Figures 6 and 7 show a sample of the comparison of monthly mean flows and standard deviations of observed and reconstituted series for Nam Mae Yuam (04011002).

It was observed that except for the main Salawin river, the average specific yields of the streams in the basin were of similar order of magnitude and ranged from 6.13 to 47.20 l/sec/sq.km. For the Salawin river, the yield was computed to be 123.28 l/sec/sq.km.. For typical stations, mean monthly flows and their percentage distributions are plotted in Figures 8 and 9. It is generally observed that August and September are the months of largest flows. More than 2/3rd of the annual flow occurs during the period from July to November.

The ratio of mean flows during peak and driest months ranges from about 6 to 30. The variability of monthly flows during rainy months was found to be higher compared to that during relatively dry months. The average annual sediment yield varied from 22.1 to 460.18 tons/sq.km. and more than 75% of the annual sediment load occurred between July and October. It was noted that the sediment yield of the Salawin river at 2.46 tons/sq.km was significantly lower.

3.3 Groundwater: Information on hydrogeology and groundwater wells in the study area is limited, partly because of the generally low potential and only limited use (mainly for drinking purpose) of groundwater at present. The geological formations in the area are made up of consolidated and unconsolidated rocks. The consolidated rocks consist of many rock types with widely different lithology, age and structural history. The consolidated rocks are not suitable for groundwater development because they occur in the mountainous area (which forms the major part) and ground water in limited quantity is stored in joints, fractures, cavities, bedding planes, decomposed zones, or contact zones with neighboring rocks.

In the study area, the unconsolidated deposits are very limited and important groundwater aquifers are only the unconsolidated sediments of alluvial and terrace deposits which occur as a narrow strip in the Mae Sariang district, where jetted and dug wells can be found. A generalized hydrogeological section of the Nam Mae Yuam sub-basin showing the unconsolidated deposits of sand, gravel, and clay of Pleistocene to Recent ages overlying bedrock of shale of Tertiary age is illustrated in Figure 10. The thickness of the unconsolidated sediments is estimated as 30-50 meter. Well yield is 5-10 m^3/hr, with good quality water.

Limited information is available about the quality of surface water as well as ground water in the study area. However, they indicate that physical, chemical and bacteriological properties of water are mostly well within the National Drinking Water Quality Standards for surface and ground water (NEB, 1989). Only in few cases, turbidity (in case of surface water during rainy season) and manganese content (in case of ground water) were found to be over the maximum allowable concentration. Thus, the existing natural conditions as well as the human impacts appear to be environmentally sound so far.

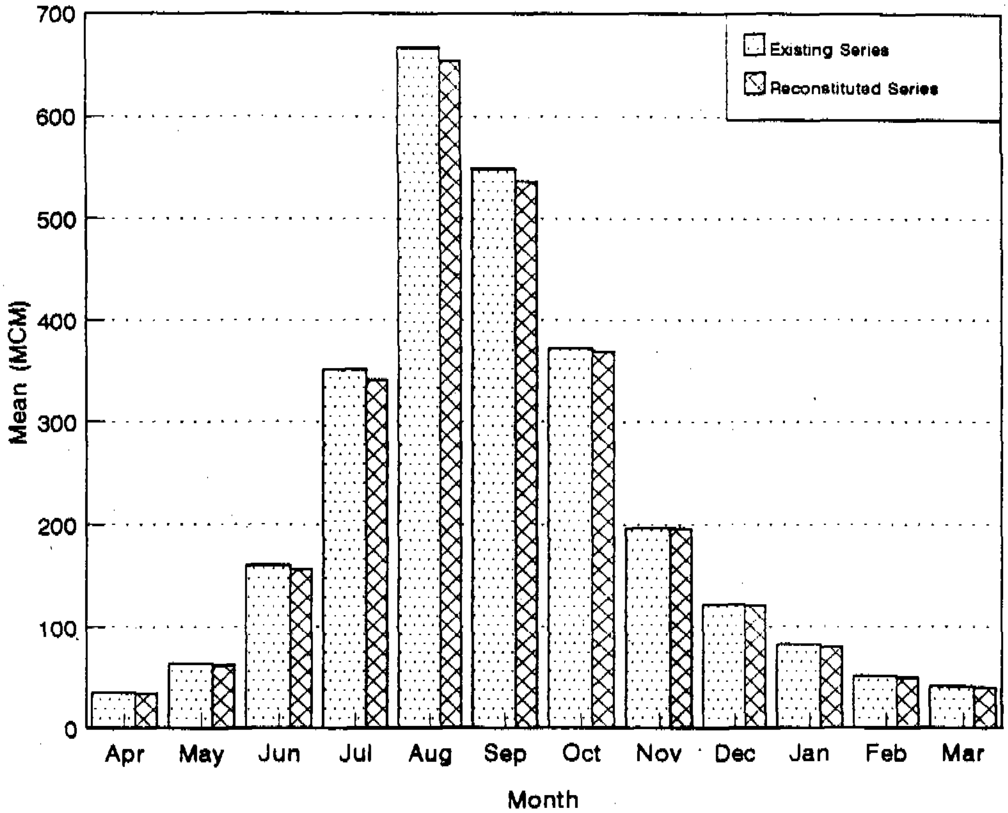

Figure 6 Monthly Mean of Observed Vs Reconstituted Stream Flow series for Nam Mae Yuam (04011002)

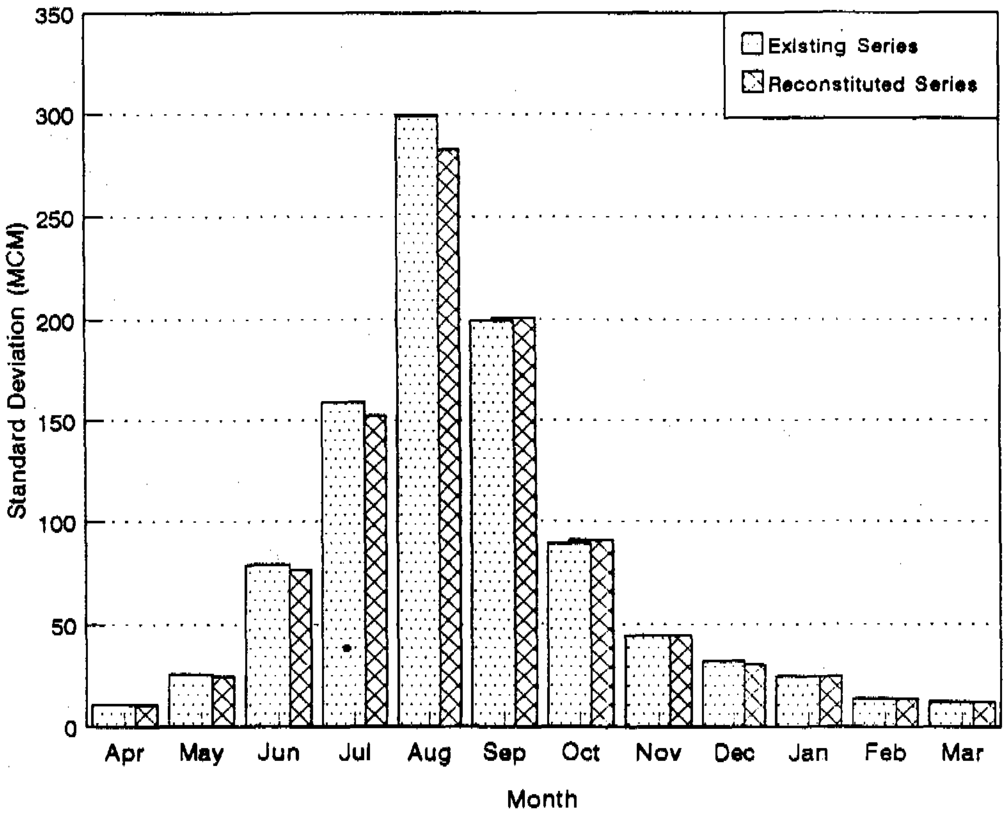

Figure 7 Standard Deviation of Existing Vs Reconstituted Stream Flow Series for Nam Mae Yuam (04011002)

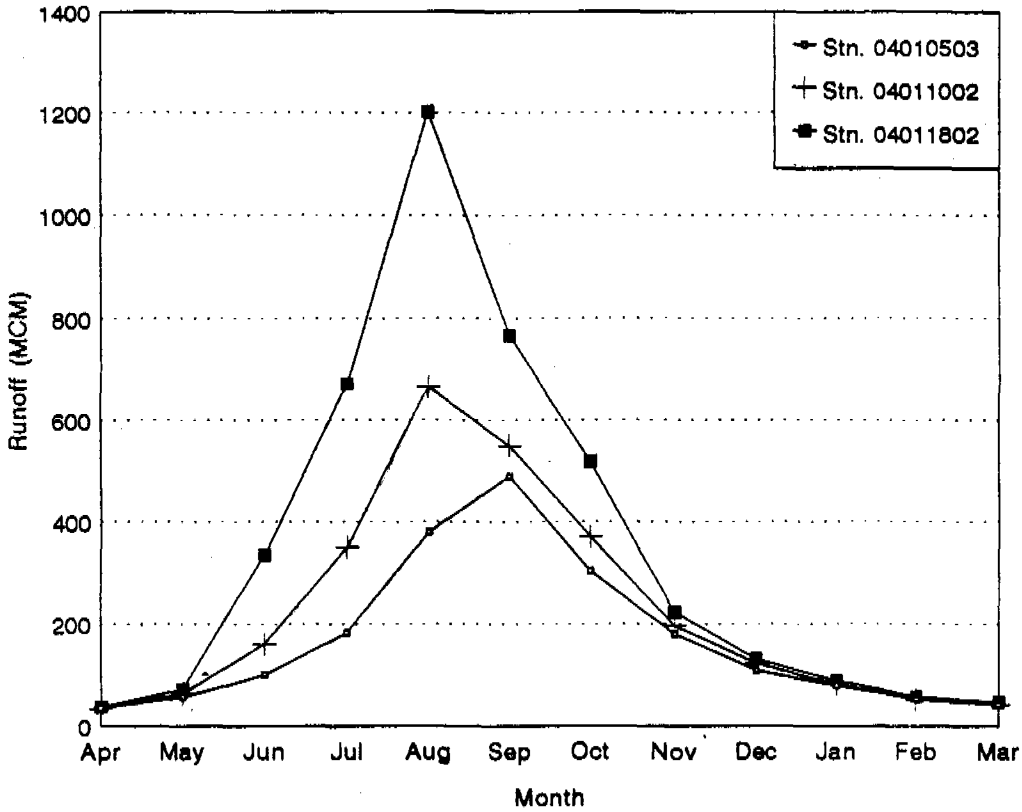

Figure 8 Mean Monthly Runoff Volumes for Typical Stations

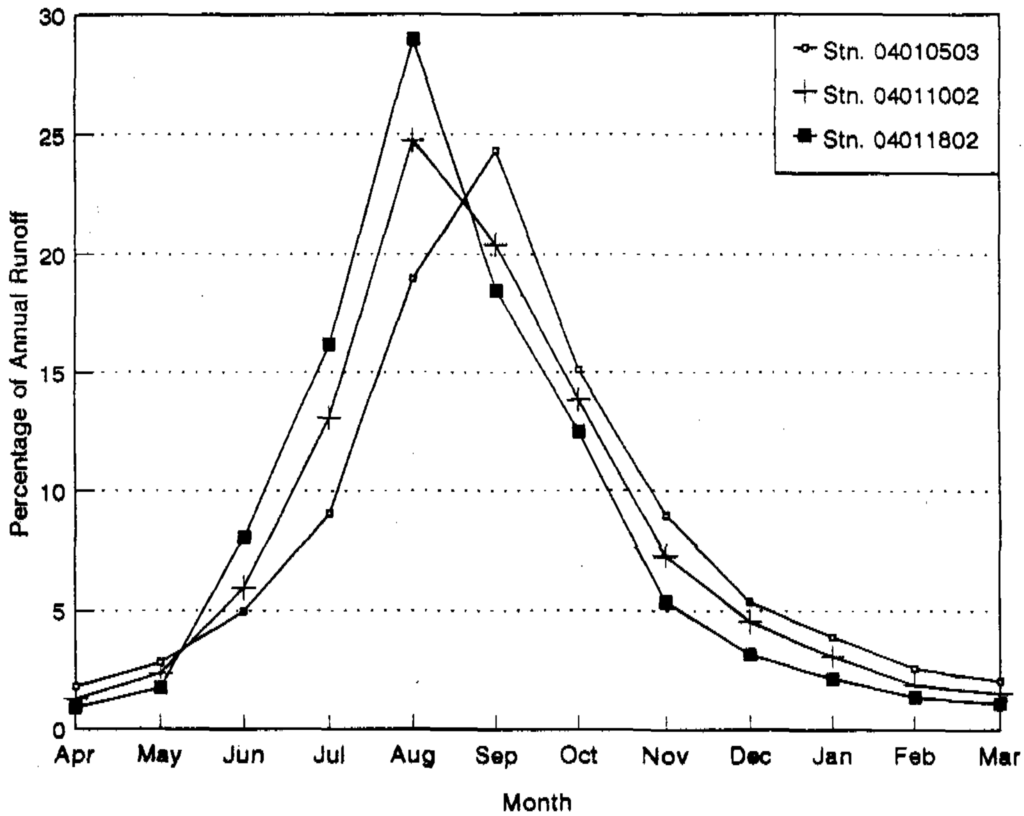

Figure 9 Percentage Distribution of Mean Monthly Runoff for Typical Stations

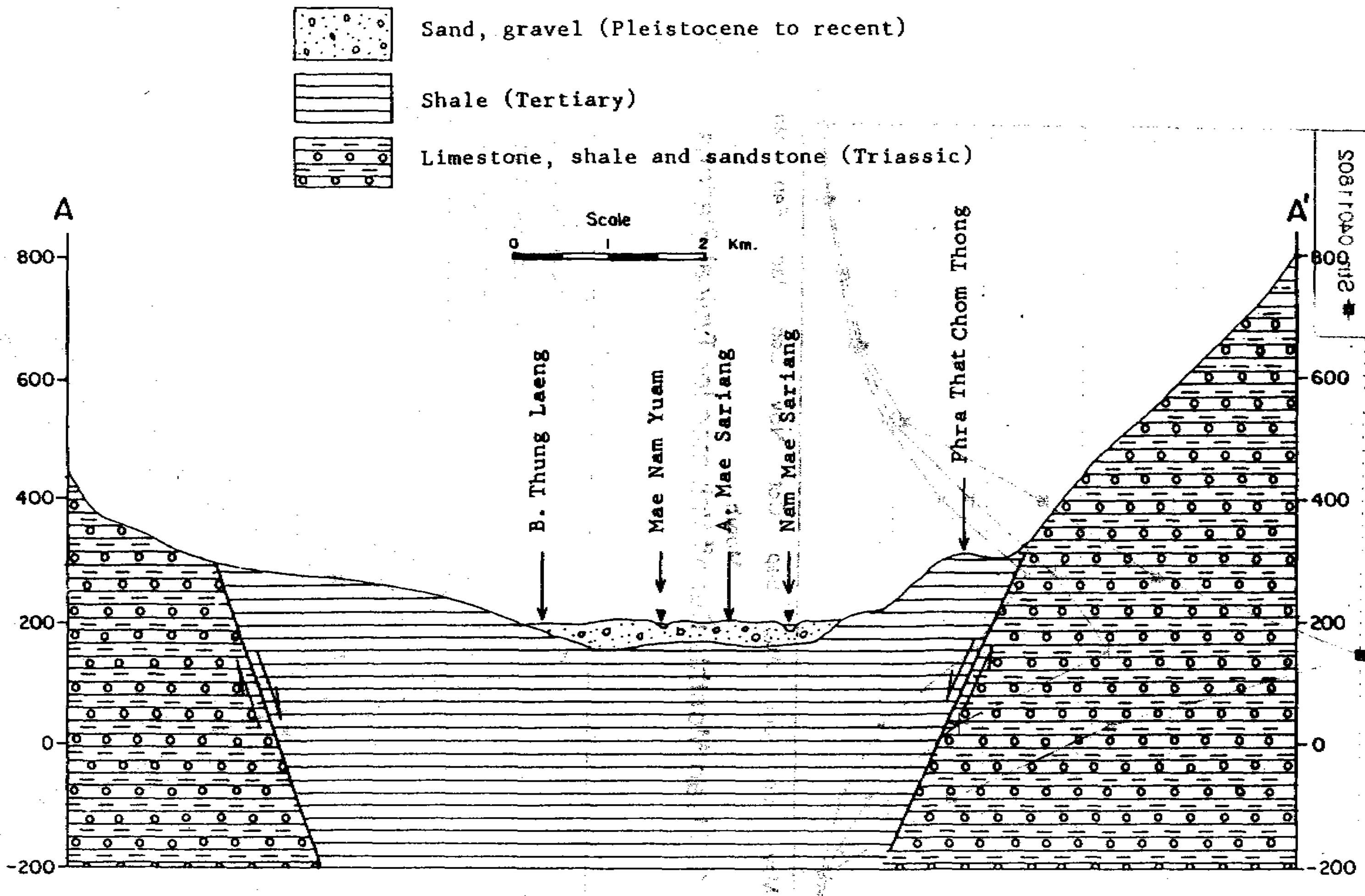

Figure 10 Generalized Hydrogeological Section of Nam Mae Yuam Sub-basin

4. WATER DEMANDS

4.1 Irrigation: Though there are no large scale irrigation projects in the Salawin basin, irrigation is and will continue to be the principal use for water in the basin. Irrigation water demand were calculated for existing as well as short term (1994-1996) and long term (1997-2006) conditions. Irrigation water requirements were estimated based on the guidelines provided by NESDB, starting from the information on sub-basinwise irrigation area for the three cases, each divided into the 3 types of irrigation schemes, namely, medium scale, small scale and pumping irrigation systems. Areas were also included for the three main sub-basins (viz. Pai, Yuam and Moei).

It is estimated that about 30,234 ha and 9,111 ha are currently irrigated during wet season and dry season respectively. Medium scale irrigation projects cover only about 19% and 38% in wet and dry season respectively while the rest come under small scale and pumped projects. For medium scale and pumping projects, dry season cropping intensity was considered to be 60% of the wet season (100%) values, whereas for small scale the corresponding figure was taken as 20%. These values for cropping intensities are based upon the information on existing projects and field enquiries. Considering the irrigable land and potential projects in the study area, the total proposed irrigation areas in 1996 and 2006 are 34,149 and 57,155 ha respectively. The proportion of medium scale, small scale and pumping schemes are respectively 19.9%, 72.9%, and 7.2% for 1996, and 41.4%, 53.3% and 5.3% for 2006.

In order to properly reflect the system type and agro-meteorological characteristics of the study area, six models were considered for calculating net irrigation water requirements for an unit area. Two system types (i.e. medium scale or pumping schemes with 160% cropping intensity and small scale schemes with 120% cropping intensity) and three main sub-basins (i.e. Pai, Yuam, and Moei) were considered. The same cropping calendar for rice and upland crop (mainly soybean) was assumed.

The calculations were done based on the reference crop evapotranspiration computed from the FAO Modified Penman method. Crop coefficients and effective rainfalls for rice and upland (soybean) crops were used as suggested by RID. Percolation (at the rate of 2 mm/day) was considered only for rice. Additionally, total land preparation and nursery bed preparation requirements of 240 mm and 400 mm respectively were considered for rice. Meteorological parameters (including rainfall) of the Mae Hong Son, Mae Sariang and Mae Sot index stations were considered for Pai, Yuam and Moei sub-basins respectively. Different irrigation efficiencies were considered for wet and dry seasons, and for existing (15%, 30%), short-term (15%, 30%) and long-term (30%, 50%) planning periods. It is noted that these values are quite conservative and the long-term values consider improvement in efficiencies.

The information on irrigation demand are summarized in Table 2. Annual gross water demands were estimated to be 616.927 MCM, 696.741 MCM and 635.03 MCM respectively for existing (1993), short-term (1996) and long-term (2006) conditions. Irrigation demands exist mainly in sub-basins 2, 5, 8, 10, 16, 17 and 18. It is noted that, in spite of more irrigation area, the long-

Table 2 : Summary Information on Total Irrigation Water Demand in MCM

Items	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
(i) Existing Condition (1993)													
(a) Medium Project	13.50	10.41	0	0	0	24.18	25.81	13.68	24.92	12.65	2.48	9.82	137.45
(b) Small Project	17.10	13.37	0	0	0	96.09	102.33	53.68	95.52	42.16	2.84	11.36	434.44
(c) Pumping Project	4.51	3.48	0	0	0	7.95	8.33	4.31	8.26	4.11	0.75	3.35	45.04
Total	35.11	27.25	0	0	0	128.22	136.47	71.68	128.69	58.91	6.07	24.53	616.93
(ii) Short term (1996)													
(a) Medium Project	15.44	12.02	0	0	0	28.49	30.67	16.38	28.71	13.80	2.85	11.10	159.47
(b) Small Project	18.76	14.72	0	0	0	106.60	113.94	59.98	105.12	45.44	3.13	12.45	480.13
(c) Pumping Project	5.57	4.36	0	0	0	10.32	11.00	5.79	10.35	4.74	0.95	4.06	57.15
Total	39.78	31.10	0	0	0	145.40	155.61	82.15	144.18	63.99	6.93	27.61	696.74
(iii) Long term (2006)													
(a) Medium Project	34.53	26.19	0	0	0	48.64	50.70	26.25	52.26	28.63	6.07	25.79	299.07
(b) Small Project	13.74	10.78	0	0	0	65.05	69.51	36.58	64.14	27.68	2.28	9.10	298.86
(c) Pumping Project	4.07	3.23	0	0	0	6.51	6.98	3.69	6.35	2.66	0.68	2.94	37.10
Total	52.34	40.20	0	0	0	120.20	127.19	66.51	122.75	58.98	9.03	37.83	635.03
(iv) Long term (2006) with existing efficiency													
(a) Medium Project	57.53	43.64	0	0	0	97.24	101.36	52.47	104.48	57.26	10.12	42.97	567.06
(b) Small Project	22.90	17.97	0	0	0	130.10	139.02	73.15	128.28	55.36	3.81	15.17	585.76
(c) Pumping Project	6.78	5.38	0	0	0	13.01	13.96	7.37	12.70	5.33	1.13	4.89	70.56
Total	87.21	66.99	0	0	0	240.36	254.33	132.99	245.47	117.95	15.06	63.03	1223.38

term irrigation demand is less than the short-term demand due to significantly higher irrigation efficiencies assumed to be achieved in the long run. With existing efficiencies (i.e. considering no improvement), however, it is found that the annual gross water demand of 1223.38 MCM is much higher. About 85% and 15% of the annual demand occur in wet season (paddy) and dry season (mainly soybean) respectively for both existing and short term conditions whereas the corresponding figures for long-term condition are 78% and 12%.

4.2 Domestic and Other Uses: Estimates of population and its characteristics are required for estimating domestic water demands. The total population in the study area in 1992 was 450,815, with the rural/urban ratio being approximately 4.2 and the growth rate 1.5%. Based on the available information and estimates, the consumption rates for domestic purpose in the study area were considered as 50 and 175 liters per capita per day respectively in rural and urban areas. Table 3 shows the summary of gross domestic water demand for different sub-basins in the study area. Gross domestic water demands in 1993 and 2006 are 11.77 MCM and 14.29 MCM.

Similarly, based on certain criteria for water consumption rates and the growth rates assumed for other sectors of economy, water demands were estimated for existing condition (1993) as well as for short term (1994-1996) and long term (1997-2006) planning periods. For industries, the annual growth rate is assumed to be 20% while the water consumption is taken as 10 m^3/day/factory. For tourism, the growth rate is considered as 1.8% and 1.5% for Mae Hong Son and Tak respectively. Tourists are assumed to stay 3 days on an average consuming 0.615 m^3/day/capita. The growth rates for cattle, buffalo, swine, duck/chicken, goose, goat/horse, elephant and sheep are considered respectively 3.2, -2.3, 3.1, 5.2, 5.2 11.4, -4.8 and 3.2 percent. The respective figure for water consumption rates are 50, 50, 20, .015, .015, 40, 70 and 15 liters/day. Table 3 presents the summary of water demands for industry, tourism and livestock in various sub-basins. The water demands for industry, tourism and livestock are 1.98 MCM, 0.52 MCM and 1.95 MCM respectively in 1993, and 21.18 MCM, 0.65 MCM and 2.45 MCM respectively in 2006.

The other major water use activities for which water demands exist are hydropower generation, interbasin transfer and environmental quality preservation, particularly on downstream. Hydropower generation is a non-consumptive use of water and major hydropower projects are not likely to be developed in the basin until the year 2006. However, the Mae Lamao - Bhumibol interbasin diversion project to augment the water supply to the central region of the country from the Bhumibol dam/reservoir system looks feasible (Newjec et.al,1993).

As suggested by the NESDB guidelines, minimum water necessary to maintain low flow conditions and environmental quality at various control points have been estimated based on the average annual low flows. Because this is a very conservative assumption, probability of shortage in meeting these minimum flow requirements, particularly during dry season, is likely to be high. Since the available information on water quality conditions indicate sound natural conditions and negligible human impacts, water demands for environmental quality control are not expected to be great.

Table 3: Water Demands (MCM) for Various Purposes

Year	Subbasin																	Total
	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
(i) Domestic																		
1993	0.475	0.067	0.108	1.068	0.221	0.264	0.353	0.141	1.498	0.402	0.586	0.221	0.221	0.041	3.467	0.838	1.808	11.779
1996	0.496	0.070	0.113	1.117	0.231	0.276	0.369	0.147	1.567	0.420	0.612	0.231	0.231	0.043	3.627	0.877	1.891	12.318
2006	0.576	0.082	0.132	1.296	0.268	0.321	0.428	0.171	1.813	0.487	0.711	0.268	0.267	0.050	4.210	1.017	2.194	14.291
(ii) Industrial																		
1993	0.000	0.000	0.090	0.448	0.358	0.000	0.000	0.000	0.078	0.050	0.027	0.081	0.077	0.077	0.258	0.366	0.072	1.980
1996	0.000	0.000	0.155	0.774	0.619	0.000	0.000	0.000	0.135	0.086	0.046	0.139	0.133	0.133	0.445	0.632	0.125	3.421
2006	0.000	0.000	0.958	4.790	3.832	0.000	0.000	0.000	0.835	0.534	0.287	0.863	0.821	0.821	2.756	3.913	0.773	21.184
(iii) Tourism																		
1993	0.017	0.009	0.034	0.089	0.064	0.004	0.009	0.001	0.008	0.005	0.003	0.008	0.008	0.008	0.104	0.156	0.000	0.525
1996	0.018	0.009	0.036	0.094	0.067	0.004	0.009	0.001	0.008	0.005	0.003	0.008	0.008	0.008	0.109	0.163	0.000	0.552
2006	0.022	0.011	0.043	0.112	0.080	0.005	0.011	0.001	0.010	0.007	0.004	0.010	0.010	0.010	0.126	0.189	0.000	0.650
(iv) Livestock																		
1993	0.071	0.036	0.062	0.205	0.108	0.056	0.192	0.081	0.165	0.045	0.110	0.150	0.088	0.054	0.165	0.361	0.006	1.953
1996	0.073	0.037	0.064	0.210	0.109	0.057	0.197	0.082	0.167	0.045	0.114	0.155	0.089	0.053	0.179	0.391	0.006	2.026
2006	0.085	0.042	0.074	0.236	0.120	0.064	0.226	0.090	0.181	0.046	0.133	0.183	0.097	0.053	0.250	0.561	0.008	2.447

5. SIMULATION MODEL ANALYSIS

In order to study the water supply and water demand relationship for existing as well as future conditions and to determine the maximum potential for water resources development in the basin, the HEC-3 "Reservoir System Analysis for Conservation" developed by the Hydrologic Engineering Center of the US Army Crops of Engineers in 1981 was used (HEC, 1981). The model simulates the behavior of a water resources system for such conservation purposes as water supply, navigation, recreation, low-flow augmentation and hydroelectric power. HEC-3 model is popular in Thailand at present.

For the analysis purpose, the whole basin in the study area was conceptualized by dividing into three independent main sub-basins (viz. Nam Mae Pai, Nam Mae Yuam, Mae Nam Moei) and a combined sub-basin consisting of two smaller independent sub-basins (viz. Huai Mae Tae Luang and Nam Mae Ngae). HEC-3 was used to run each model configuration for the three water demand scenarios (i.e. existing case - 1993; short term case - 1996; and long term case - 2006) and 27 years of streamflow data reconstituted using the HEC-4 model. The simulation model also considered such details as the minimum flow constraints, return flow components, proposed project characteristics, and diversion requirements at various specified locations. The sub-models were conceptualized and run individually such that, when combined, they yielded the water balance for the whole study area.

For the long-term development by the year 2006, Table 4 gives the summary of water balance for the whole basin. The analysis of long-term situation in 2006 corresponds nearly to full development of the basin at the end of the Ninth National Economic and Social Development Plan (2002-2006). In order to investigate the conservative case, a second long-term scenario was also studied through the HEC-3 model using the presently existing efficiency values. There are some dams/reservoirs and interbasin water diversion projects proposed in the study area which will be realized only during the full development stage of the basin beyond the year 2006. One exception is Mae Lamao-Bhumibol Interbasin Diversion Project in Moei sub-basin, which has been already investigated at the feasibility study level by Newjec et.al.(1993) for EGAT. This project, found to be feasible and anticipated to be constructed during 1998-2002, is considered in the analysis.

For the long-term case, as in the existing and short term cases, some shortages - particularly in meeting the required minimum flows - occur in sub-basins 2, 5, 10 and 16. For the basin as a whole, about 7.2% and 0.6% of the average annual inflow of 8565.67 MCM are diverted for irrigation and domestic/industrial water supply. About 6.2% is actually consumed while 89.8% flows out of the basin. The shortages in irrigation and low flow maintenance are only 2.6% and 3.8% but the shortage in interbasin transfer is almost 57.0%. When irrigation efficiencies are assumed to be the same as existing, the results are similar but are affected by almost two-fold increase in irrigation demands. For the whole basin, about 12.9%, and 0.6% of the average annual inflow are diverted for irrigation and domestic/industrial water supply respectively. The consumptive use and outflow account for about 11.7% and 88.3%. In this case also, the shortages in irrigation and required minimum flows are 9.9% and 5.6% but the shortage in interbasin diversion is 61.9%. The shortages occur mainly during the dry season. Relatively high interbasin diversion shortage is due to the fact that the diversion requirement is based upon the proposed tunnel capacity of 25 m^3/sec.

Table 4 : Summary of Water Balance Study (MCM) : Long Term (2006)

Description	Nam Mae Pai Sub basin Total	Nam Mae Yuam Sub basin Total	Mae Nam Moei Sub basin Total	Independent Sub basin Total	Whole Basin
Inflow	2406.63	2769.90	2412.74	976.40	8565.67
Demand	184.17	153.52	1135.87	4.98	1478.54
(a) Irrigation	164.30	143.10	325.52	1.92	634.84
(b) Others	19.87	10.42	21.95	3.06	55.30
(c) Trans Basin	–	–	788.40	–	788.40
Diversion	176.96	151.00	680.92	4.98	1013.86
(a) Irrigation	157.09	140.58	319.72	1.92	619.31
(b) Others	19.87	10.42	21.95	3.06	55.30
(c) Trans Basin	–	–	339.25	–	339.25
Shortage	8.02	2.52	454.95	0.00	465.49
(a) Irrigation	8.02	2.52	5.80	0.00	16.34
(b) Others	0.00	0.00	0.00	0.00	0.00
(c) Trans Basin	–	–	449.15	–	449.15
Req. Minimum Flow	334.28	310.44	268.46	114.75	1027.93
Actual Min. Flow	328.43	309.26	237.24	114.27	989.20
Shortage (Min. Flow)	5.85	1.18	31.22	0.48	38.73
Return Flow	25.70	12.91	102.14	0.75	141.49
Consumptive Use	151.26	138.09	239.53	4.23	533.12
Out Flow	2255.37	2631.81	1833.96	972.17	7693.30

6. SUMMARY AND CONCLUSIONS

An assessment of water resources as well as water demands in the Salawin river basin in Thailand within the context of national level water resources planning has been presented. The paper only considers the portion of the international basin lying within the territory of Thailand. The study area covers about 17,920 sq.km. of mountainous and relatively fragile watershed in the remote northwestern part of the country. It has been divided into 18 sub-basins for the purpose of analysis.

Hydrological analyses have shown that the monsoon influenced rainfall and surface water resources situations are fairly good, though with a distinct dry season. Ground water resources is limited. Quality of both surface and ground water is good. Water demands for various uses such as irrigation, domestic and industrial water supply, livestock and tourism, hydropower, and environmental control during the planning period (1993-2006) are relatively low. Various water supply versus water demand scenarios analyzed by using the HEC-3 river basin simulation model have indicated that the water supply and demand situation in the Salawin river basin appears to be favorable in the foreseeable future until the year 2006. In fact, the situation looks very good even beyond 2006 for the full potential development of the basin. Hence, the proposals to divert water from the Salawin river basin to the adjoining basins in Thailand, where there are acute water shortage problems, appear to be reasonable provided necessary attention is paid to the socio-economic and environmental impacts of such interbasin transfer projects.

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Techno-political Decision-making for Water Resources Development: The Jordan River Watcrshed

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ABSTRACT

Discussions on water resources development generally focus on a variety of technical options, often without considering the potential political repercussions of each option. This paper incorporates both technical and political considerations in a 'techno-political' decision-making framework. Water resources development alternatives are then examined to evaluate their respective priorities for development in Israel, Palestine, and Jordan, which are the major riparians of the Jordan River. Particular account is taken of the Middle East peace negotiations, and consequent political changes. Each proposal is designed to provide incentives for sharing resources and benefits among the riparian states.

ABRÉGÉ

Les débats sur le développement des ressources hydrologiques se concentrent généralement sur une variété d'options techniques qui souvent ne prennent pas en considération les répercussions politiques possibles. Cette communication incorpore les considérations techniques et politiques dans un [decision-making?] cadre "techno-politiques." Alternatives pour le développement des ressources hydrologiques sont étudiées afin d'évaluer les priorités de développement en Israël. en Palestine, et en Jordanie, c'est-à-dire les principaux pays riverains du Jourdain. Aussi, cette communication tient particulièrement compte non seulement des négotiations pour la paix au Moyen Orient, mais aussi des suites des changements politiques. Chaque proposition est conçue pour fournir des motifs qui encourageraient les pays riverains à partager ces ressources et des lors à en bénéficier aussi.

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1. INTRODUCTION[3]

The accord between Israel and the Palestine Liberation Organization (PLO) in Oslo on 13th September 1993, produced a Declaration of Principles which included proposals for an inter-state regional economic development plan (Israel/PLO, 1993). Regional economic development was conceived as a key element to sustaining the peace process in the region (see Figure 1). Similarly, regional watershed development was emphasized in an agenda for cooperation, signed between Jordan and Israel in June, 1994.

This paper examines regional water projects, considering both technical and political aspects of viability. Our focus is "techno-political" considerations of non-conventional water-energy development alternatives for each of the two inter-state regions -- the Dead Sea and Aqaba. Our considerations are in the context of sharing resources and benefits, taking into account the next possible multi-lateral peace agreement among Israel, Palestine,[4] and Jordan. Syria, Lebanon, Egypt and Saudi Arabia could also share resources and benefits from some of these schemes.

2. WATER AND ENERGY AS KEY ISSUES FOR REGIONAL DEVELOPMENT

2.1 Hydro-Political Positions[5]

Before investigating specific projects, the major hydropolitical issues facing each political entity are examined:

Israeli water issues

Sustainable yield of renewable fresh waters in Israel is approximately 1,500 x 10^6 m^3 per annum. Israel had already exceeded this level by the early 1970s, and had to cut 29% from its national water budget from 1,987 x 10^6 m^3 in 1987 to 1,420 x 10^6 m^3 in 1991 due to severe drought. Israel accomplished this without losing net agricultural product or economic growth. Overall water savings in the agricultural sector was 40% during the same period -- from 1,434 x 10^6 m^3 in 1986 to 875 x 10^6 m^3 in 1991.

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4 We use the term, "Palestine" to refer to the West Bank and Gaza, with no pre-disposition to the eventual outcome of the Middle East peace talks.

5 For more in-depth discussions of both the hydro-political positions and the technical and policy options available to the riparians of the Jordan basin, the reader is referred to Murakami (1994) and Wolf (1994).

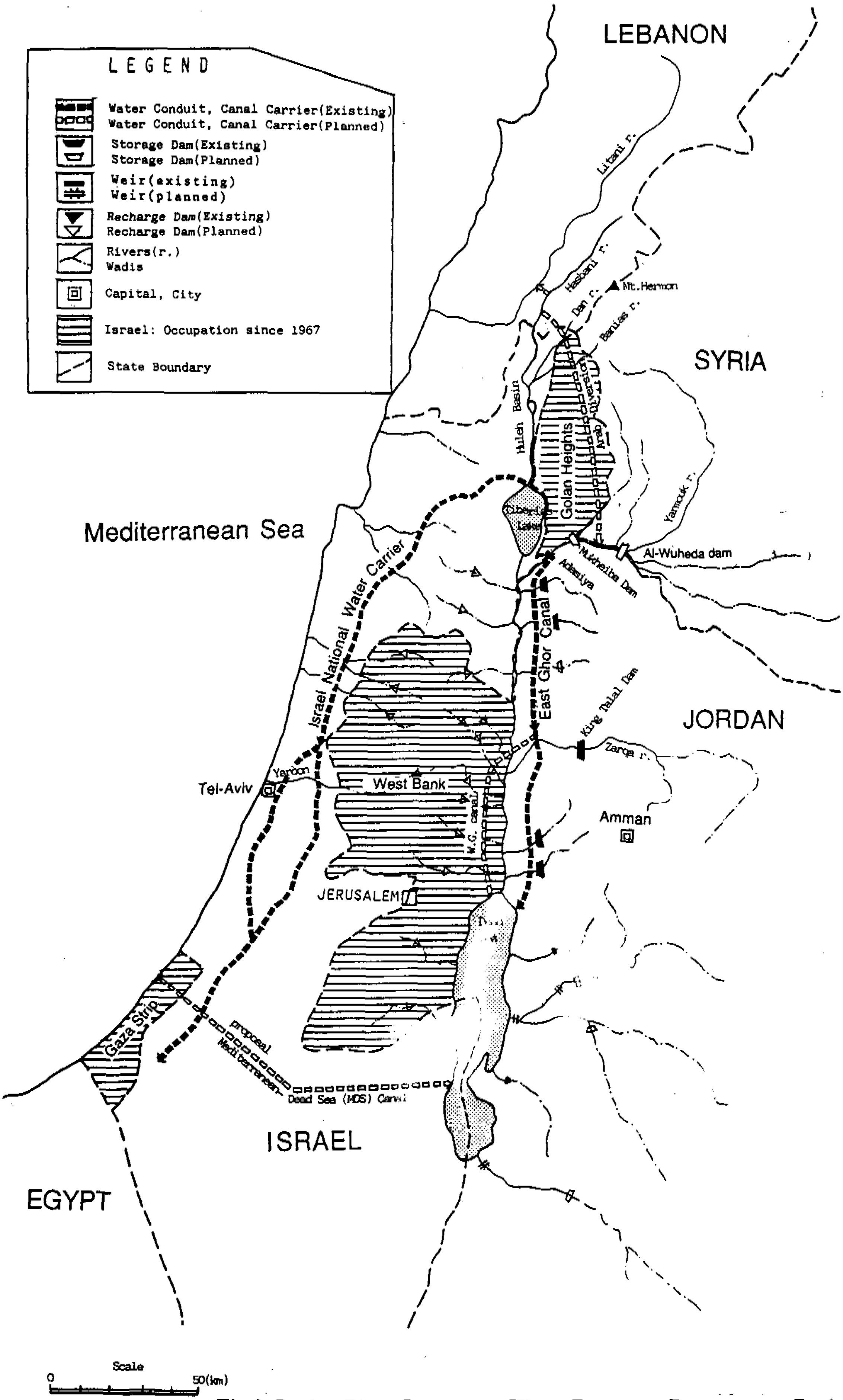

Fig.1 Jordan River System and Water Resources Development Project

Palestinian water issues

Israel took control of the West Bank in 1967, including the recharge areas for aquifers which flow west and north-west into Israel (at about 320 x 10^6 m^3/yr. and 140 x 10^6 m^3/yr. respectively) and east to the Jordan Valley (about 125 x 10^6 m^3/yr.). The entire renewable recharge of these first two aquifers is already being exploited and the third is close to being depleted as well. Total consumption within the West Bank is 35 x 10^6 m^3/yr., mostly from wells, for Israeli settlements; and 115 x 10^6 m^3/yr., from wells and cisterns, for Palestinians. Israel is dependent on the West Bank for a total of some 430 x 10^6 m^3 per annum of its water supply out of a total 1,420 x 10^6 m^3, which accounts for 30% of the annual water potential. Because any overdraft would result in saltwater intrusion along Israel's coastal plain, or eventually even into the mountain aquifers, Palestinian water usage has been severely limited by the Israeli authorities. Palestinians, on the other hand, have claimed first rights to all of the ground- and surface-water which originates on the West Bank, and have objected to Israeli controls. Palestinians were also to receive 70-170 x 10^6 m^3/yr. from the Jordanian share of the Johnston negotiations of 1953-55.

Gaza is probably the most desperate political entity in the region hydrographically. Completely dependent on the 60 x 10^6 m^3/yr. of annual groundwater recharge, Gazans currently use approximately 95 x 10^6 m^3/yr. The difference between annual supply and use is made up by overpumping in the shallow coastal aquifer, resulting in dangerous salt-water intrusion of existing wells and ever-decreasing per capita water availability, which is already the lowest in the region.

Jordanian water issues

Jordan will have exhausted its renewable fresh water resources of 870 x 10^6 m^3/yr very soon. Only two major tributaries of the Jordan river system have not been fully developed -- Wadi Mujib, with an annual flow of 78 x 10^6 m^3, and the Yarmuk, with annual flood flows of 168 x 10^6 m^3 (World Bank 1988). Wadi Mujib, which is the third largest tributary flowing into the Dead Sea, has no inter-state riparian complications as does the Yarmuk. One current question on sustainable water development in Jordan is whether it can afford to continue to develop fossil groundwater in deep sandstone aquifers, like the Disi, and if so then "for what?" and "for how long?".

2.2 Energy-Water Issues

Energy issues relating to water resources are critical matters in the development of non-oil producing countries like Israel, Palestine, and Jordan. These countries are the major riparians of the Jordan river system, and all have increasing demand for desalination and the reuse of treated wastewater, both of which consume substantial amounts of energy.

The current energy sources in the region are heavily dependent on crude oil. Israel, for example, consumes 2.5 million tons of oil and 2.3 million tons of oil-equivalent of coal for the generation of electricity. The annual production of electricity amounted to 20.9 billion kWh at an installed capacity of 5,835 MW in 1991 (State of Israel 1992). Because of its level of development, Israel is investigating the option of replacing its steam power generating system with nuclear by stages into the 21st century. A significant deficit in peak power supply has been a long standing problem, while substantial off-peak electricity is being wasted. Although international networking of the electric supply is being discussed between adjoining states including Egypt, no alternatives have been suggested other than building a new pumped-storage unit and/or gas turbine generating units.

A developed country's energy needs are closely related to its water supply, which can consume substantial electricity to move water from its sources to where it will be used. Pumping demand in Israel, for example, amounted to 1,528 kVA in 1991 (State of Israel 1992), or 30% of total expenditures on water supply by Mekorot, the national water development company. Taking into account recent advances in desalination, Israel is planning to introduce large-scale seawater desalination by the year 2000. Although this is likely to be dependent on low energy types of reverse osmosis, the energy cost will still be 50-60% of the total. Consequently the potential use of off-peak electricity will be a key element in minimizing the cost of water management and operation.

3. TECHNO-POLITICAL DECISION-MAKING

All of the technical and policy options available to any watershed reaching the limits of its water supplies are listed in Figure 2. Once the technical and policy options are known, the next, and probably the most crucial, step is to develop a method for evaluating the options against each other; that is, to create a hierarchy of viability. Many disciplines provide their own version of viability. Where an engineer might ask, "Can it be done?"; an economist might add, "At what cost?" A political analyst could suggest, "Is it politically feasible?"; and anyone environmentally aware might counter, "Should it be done at all?"

One problem with these varied standards of viability is that they often measure at cross-purposes, arriving at differing or even contradictory conclusions. Dinar and Wolf (1991), for example, evaluate a potential Nile to Jordan basin water transfer, both in terms of economic and political viability. Their findings using each standard were in diametric opposition to each other: whereas an economic analysis suggested greater payoffs for larger coalitions of states cooperating, a political investigation showed that the likelihood of such coalitions actually forming decreased as the size of the coalition increased, and that the most likely action was no cooperation whatsoever.

WATER MANAGEMENT OPTIONS TO INCREASE SUPPLY OR DECREASE DEMAND

UNILATERAL OPTIONS

DEMAND

- Population control.
- Rationing.
- Public awareness.
- Allow price of water to reflect true costs (including national water markets).
- Efficient agriculture, including:
  - Drip irrigation
  - Greenhouse technology
  - Genetic engineering for drought and salinity resistance

SUPPLY

- Wastewater reclamation.
- Increase catchment and storage (including artificial groundwater recharge).
- Cloud seeding.
- Desalination.
- Fossil aquifer development.

COOPERATIVE OPTIONS

- Shared information and technology.
- International water markets to increase distributive efficiency.
- Interbasin water transfers.
- Joint regional planning.

Fig. 2

What we suggest here is a unified approach to overall viability which incorporates established measures for technical, environmental, economic, and political viability. Technical viability measures the physical parameters of a system or proposal -- how much water might be produced? what is the quality? how reliable is the source? These physical parameters have well-defined quantitative values which might be used as indicators of viability. Quantity might be measured as volume of water produced by a project within a year, for example. Likewise, quality might be evaluated in parts per million of total salinity or particular pollutants, and a value for flux in a natural system or down-time in a technological project might be an indicator for reliability. Relative environmental degradation can also be evaluated quantitatively, if impact assessments are performed uniformly between projects.

Economic viability has two aspects -- financial, which measures the chances of acquiring financing for a project (often, but not always related to the amount of capital required), and efficiency. For relative water projects, one might use the results of a benefit/cost analysis and use the resulting net present value of benefits as a measure or, more directly, the cost per unit water which would result from each project. An important economic point is that costs are not fixed over time. A 'resource depletion curve' for any project would show at what rate the utility, or value, of a unit of water would begin to drop, and consequently, what the most efficient rate of development would be.

The most tenuous measure is political viability. To incorporate this important parameter in an integrated model, one must use a relative scale for a value which is difficult to quantify. While we recognize the general lack of enthusiasm for quantitative political analysis for its necessarily subjective nature (see Ascher 1989, for a good critique), we recommend the inclusion of results of a process such as the PRINCE Political Accounting System. Coplin and O'Leary (1976) describe the method of incorporating each player's "position," "power," and "salience," for any of a number of policy options to arrive at a relative ranking of political viability. In Coplin and O'Leary (1983), they extend the process to provide an absolute measure of the likelihood of a policy action taking place.

Two other qualitative measures might be used for political viability. For projects within a country, how well a proposal "fits" with national goals might be evaluated. Population control, for example, which might be successful in western Europe or the United States, runs counter to both Israeli and Palestinian interests in numerical superiority. International projects might be determined in terms of relative measures for "equity" of project costs and water distribution, and "control" by each political entity of its own major water sources.

If the resources are available to perform a detailed feasibility study, the results can be described quantitatively. Listed below are the proposed measure of viability, followed by the possible quantitative standards which might be used:

-- Engineering

- quantity (eg. x 10^6 m^3/yr.);
- quality (eg. ppm salinity or pollutants);
- reliability of source (eg. std. deviation of flux);

-- Environmental

- environmental impact (eg. detail of potential damage);

-- Economic

- financial (capital necessary to finance project);
- efficiency; (cost per unit of water, or net present value of benefits);

-- Political

- political probability from PRINCE model; or equity of project cost and water distribution and control of source by each entity.

More often than not, the detailed data necessary for a quantitative evaluation are not available. In that case, two options exist. The first is to substitute qualitative values: +, 0, -, for example, representing good, neutral, or poor; adequate for a preliminary analysis. We can then evaluate any possible option qualitatively with each measure of viability. By examining the results, we should get a sense of which options are more viable than others, and why. It should be remembered that these results are for a particular geographic location, and for a single point in time.

Although a column is provided for a measure for "overall viability," it is recommended that, if the column is used at all, it is used with tremendous caution. First, each measure does not necessarily have equal weight, and each was arrived at with both some subjectivity and some uncertainty. Adding or multiplying across would therefore only compound and accumulate error.

Also, by leaving the measures separate, one acquires a greater sense of why options are viable, and where emphasis can be placed for the future in order to help boost viability. Public awareness, for example, has been shown to be a very cost-effective method of saving water, but the total amount which can be saved is rather small as compared to the total water budget. In contrast, unlimited water can be made available through desalination but at a relatively higher cost. The latter might change with technologic breakthroughs, but the former is likely to remain fairly constant over time.

The second option in the absence of data necessary for a quantitative assessment is to substitute iterations of "expert opinions," first described by Gordon and Helmer (1964) as the Delphi method.[6] Experts familiar with the technical and political landscapes of a particular watershed might be asked to rank available options as to their viabilities on a consistent scale. The viability measures themselves should also be weighted as to their relative importance for that particular watershed during a particular time frame. A variation on the weighting process is first described in detail in Kepner and Tregoe (1965).

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6 See Linstone and Turoff (1975) for a good summary of the strengths, weaknesses, and applications of the Delphi method; and Needham and de Loë (1990) for its applicability to water resources planning.

It should be emphasized that this evaluation process should be iterative -- repeated often to allow for the constant changes of so many of the parameters over space and time. Changes which can affect viability include:

--Technical and Environmental:

- Fluctuations in seasonal and annual water supply, as well as long-term changes due to global warming.
- Changes in water quality.
- Technical breakthroughs.
- Relative infrastructure for each party in
  - research and development,
  - storage and delivery,
- Changes in understanding of physical system.

--Economic:

- Changing priorities for funding agencies.
- Movement along the resource depletion curve.
- Expense for water resources development.
- Changes in efficiency of water use.

--Political:

- Power relationships
  - riparian position,
  - military,
  - legal (eg. clarity of water rights).
  - form and stability of government.
- Level of hostility.

The evaluation process should also allow for interaction, with on-going feedback between the disciplines, to reflect real-world influences. For example, a project with extremely positive economic results might help overcome political reluctance to enter into cooperation. Likewise, political constraints can effectively veto a project which has been judged worthwhile in terms of its technical and economic value.

4. SPECIFIC OPTIONS AND "TECHNO-POLITICAL" VIABILITIES

4.1 Perspectives of Non-conventional Water Development Alternatives

Conventional alternatives, which include surface water and groundwater development, have the highest priority in water resources planning where there are still renewable fresh waters to be developed and intricate inter-state riparian questions do not result. This ideal situation does not exist in most countries of the Middle East. After exploiting all of the renewable fresh water resources within their national boundaries, Israel, Palestine, and Jordan will have no choice except to develop transboundary waters and/or non-conventional waters. Water conservation is an important and essential issue in water management, and outside sources should not be considered until sources within the basin are being used at

4. WATER DEMANDS

4.1 Irrigation: Though there are no large scale irrigation projects in the Salawin basin, irrigation is and will continue to be the principal use for water in the basin. Irrigation water demand were calculated for existing as well as short term (1994-1996) and long term (1997-2006) conditions. Irrigation water requirements were estimated based on the guidelines provided by NESDB, starting from the information on sub-basinwise irrigation area for the three cases, each divided into the 3 types of irrigation schemes, namely, medium scale, small scale and pumping irrigation systems. Areas were also included for the three main sub-basins (viz. Pai, Yuam and Moei).

It is estimated that about 30,234 ha and 9,111 ha are currently irrigated during wet season and dry season respectively. Medium scale irrigation projects cover only about 19% and 38% in wet and dry season respectively while the rest come under small scale and pumped projects. For medium scale and pumping projects, dry season cropping intensity was considered to be 60% of the wet season (100%) values, whereas for small scale the corresponding figure was taken as 20%. These values for cropping intensities are based upon the information on existing projects and field enquiries. Considering the irrigable land and potential projects in the study area, the total proposed irrigation areas in 1996 and 2006 are 34,149 and 57,155 ha respectively. The proportion of medium scale, small scale and pumping schemes are respectively 19.9%, 72.9%, and 7.2% for 1996, and 41.4%, 53.3% and 5.3% for 2006.

In order to properly reflect the system type and agro-meteorological characteristics of the study area, six models were considered for calculating net irrigation water requirements for an unit area. Two system types (i.e. medium scale or pumping schemes with 160% cropping intensity and small scale schemes with 120% cropping intensity) and three main sub-basins (i.e. Pai, Yuam, and Moei) were considered. The same cropping calendar for rice and upland crop (mainly soybean) was assumed.

The calculations were done based on the reference crop evapotranspiration computed from the FAO Modified Penman method. Crop coefficients and effective rainfalls for rice and upland (soybean) crops were used as suggested by RID. Percolation (at the rate of 2 mm/day) was considered only for rice. Additionally, total land preparation and nursery bed preparation requirements of 240 mm and 400 mm respectively were considered for rice. Meteorological parameters (including rainfall) of the Mae Hong Son, Mae Sariang and Mae Sot index stations were considered for Pai, Yuam and Moei sub-basins respectively. Different irrigation efficiencies were considered for wet and dry seasons, and for existing (15%, 30%), short-term (15%, 30%) and long-term (30%, 50%) planning periods. It is noted that these values are quite conservative and the long-term values consider improvement in efficiencies.

The information on irrigation demand are summarized in Table 2. Annual gross water demands were estimated to be 616.927 MCM, 696.741 MCM and 635.03 MCM respectively for existing (1993), short-term (1996) and long-term (2006) conditions. Irrigation demands exist mainly in sub-basins 2, 5, 8, 10, 16, 17 and 18. It is noted that, in spite of more irrigation area, the long-

Table 2 : Summary Information on Total Irrigation Water Demand in MCM

Items	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
(i) Existing Condition (1993)													
(a) Medium Project	13.50	10.41	0	0	0	24.18	25.81	13.68	24.92	12.65	2.48	9.82	137.45
(b) Small Project	17.10	13.37	0	0	0	96.09	102.33	53.68	95.52	42.16	2.84	11.36	434.44
(c) Pumping Project	4.51	3.48	0	0	0	7.95	8.33	4.31	8.26	4.11	0.75	3.35	45.04
Total	35.11	27.25	0	0	0	128.22	136.47	71.68	128.69	58.91	6.07	24.53	616.93
(ii) Short term (1996)													
(a) Medium Project	15.44	12.02	0	0	0	28.49	30.67	16.38	28.71	13.80	2.85	11.10	159.47
(b) Small Project	18.76	14.72	0	0	0	106.60	113.94	59.98	105.12	45.44	3.13	12.45	480.13
(c) Pumping Project	5.57	4.36	0	0	0	10.32	11.00	5.79	10.35	4.74	0.95	4.06	57.15
Total	39.78	31.10	0	0	0	145.40	155.61	82.15	144.18	63.99	6.93	27.61	696.74
(iii) Long term (2006)													
(a) Medium Project	34.53	26.19	0	0	0	48.64	50.70	26.25	52.26	28.63	6.07	25.79	299.07
(b) Small Project	13.74	10.78	0	0	0	65.05	69.51	36.58	64.14	27.68	2.28	9.10	298.86
(c) Pumping Project	4.07	3.23	0	0	0	6.51	6.98	3.69	6.35	2.66	0.68	2.94	37.10
Total	52.34	40.20	0	0	0	120.20	127.19	66.51	122.75	58.98	9.03	37.83	635.03
(iv) Long term (2006) with existing efficiency													
(a) Medium Project	57.53	43.64	0	0	0	97.24	101.36	52.47	104.48	57.26	10.12	42.97	567.06
(b) Small Project	22.90	17.97	0	0	0	130.10	139.02	73.15	128.28	55.36	3.81	15.17	585.76
(c) Pumping Project	6.78	5.38	0	0	0	13.01	13.96	7.37	12.70	5.33	1.13	4.89	70.56
Total	87.21	66.99	0	0	0	240.36	254.33	132.99	245.47	117.95	15.06	63.03	1223.38

term irrigation demand is less than the short-term demand due to significantly higher irrigation efficiencies assumed to be achieved in the long run. With existing efficiencies (i.e. considering no improvement), however, it is found that the annual gross water demand of 1223.38 MCM is much higher. About 85% and 15% of the annual demand occur in wet season (paddy) and dry season (mainly soybean) respectively for both existing and short term conditions whereas the corresponding figures for long-term condition are 78% and 12%.

4.2 Domestic and Other Uses: Estimates of population and its characteristics are required for estimating domestic water demands. The total population in the study area in 1992 was 450,815, with the rural/urban ratio being approximately 4.2 and the growth rate 1.5%. Based on the available information and estimates, the consumption rates for domestic purpose in the study area were considered as 50 and 175 liters per capita per day respectively in rural and urban areas. Table 3 shows the summary of gross domestic water demand for different sub-basins in the study area. Gross domestic water demands in 1993 and 2006 are 11.77 MCM and 14.29 MCM.

Similarly, based on certain criteria for water consumption rates and the growth rates assumed for other sectors of economy, water demands were estimated for existing condition (1993) as well as for short term (1994-1996) and long term (1997-2006) planning periods. For industries, the annual growth rate is assumed to be 20% while the water consumption is taken as 10 m^3/day/factory. For tourism, the growth rate is considered as 1.8% and 1.5% for Mae Hong Son and Tak respectively. Tourists are assumed to stay 3 days on an average consuming 0.615 m^3/day/capita. The growth rates for cattle, buffalo, swine, duck/chicken, goose, goat/horse, elephant and sheep are considered respectively 3.2, -2.3, 3.1, 5.2, 5.2 11.4, -4.8 and 3.2 percent. The respective figure for water consumption rates are 50, 50, 20, .015, .015, 40, 70 and 15 liters/day. Table 3 presents the summary of water demands for industry, tourism and livestock in various sub-basins. The water demands for industry, tourism and livestock are 1.98 MCM, 0.52 MCM and 1.95 MCM respectively in 1993, and 21.18 MCM, 0.65 MCM and 2.45 MCM respectively in 2006.

The other major water use activities for which water demands exist are hydropower generation, interbasin transfer and environmental quality preservation, particularly on downstream. Hydropower generation is a non-consumptive use of water and major hydropower projects are not likely to be developed in the basin until the year 2006. However, the Mae Lamao - Bhumibol interbasin diversion project to augment the water supply to the central region of the country from the Bhumibol dam/reservoir system looks feasible (Newjec et.al,1993).

As suggested by the NESDB guidelines, minimum water necessary to maintain low flow conditions and environmental quality at various control points have been estimated based on the average annual low flows. Because this is a very conservative assumption, probability of shortage in meeting these minimum flow requirements, particularly during dry season, is likely to be high. Since the available information on water quality conditions indicate sound natural conditions and negligible human impacts, water demands for environmental quality control are not expected to be great.

Table 3: Water Demands (MCM) for Various Purposes

Year	Subbasin																	Total
	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	
(i) Domestic																		
1993	0.475	0.067	0.108	1.068	0.221	0.264	0.353	0.141	1.498	0.402	0.586	0.221	0.221	0.041	3.467	0.838	1.808	11.779
1996	0.496	0.070	0.113	1.117	0.231	0.276	0.369	0.147	1.567	0.420	0.612	0.231	0.231	0.043	3.627	0.877	1.891	12.318
2006	0.576	0.082	0.132	1.296	0.268	0.321	0.428	0.171	1.813	0.487	0.711	0.268	0.267	0.050	4.210	1.017	2.194	14.291
(ii) Industrial																		
1993	0.000	0.000	0.090	0.448	0.358	0.000	0.000	0.000	0.078	0.050	0.027	0.081	0.077	0.077	0.258	0.366	0.072	1.980
1996	0.000	0.000	0.155	0.774	0.619	0.000	0.000	0.000	0.135	0.086	0.046	0.139	0.133	0.133	0.445	0.632	0.125	3.421
2006	0.000	0.000	0.958	4.790	3.832	0.000	0.000	0.000	0.835	0.534	0.287	0.863	0.821	0.821	2.756	3.913	0.773	21.184
(iii) Tourism																		
1993	0.017	0.009	0.034	0.089	0.064	0.004	0.009	0.001	0.008	0.005	0.003	0.008	0.008	0.008	0.104	0.156	0.000	0.525
1996	0.018	0.009	0.036	0.094	0.067	0.004	0.009	0.001	0.008	0.005	0.003	0.008	0.008	0.008	0.109	0.163	0.000	0.552
2006	0.022	0.011	0.043	0.112	0.080	0.005	0.011	0.001	0.010	0.007	0.004	0.010	0.010	0.010	0.126	0.189	0.000	0.650
(iv) Livestock																		
1993	0.071	0.036	0.062	0.205	0.108	0.056	0.192	0.081	0.165	0.045	0.110	0.150	0.088	0.054	0.165	0.361	0.006	1.953
1996	0.073	0.037	0.064	0.210	0.109	0.057	0.197	0.082	0.167	0.045	0.114	0.155	0.089	0.053	0.179	0.391	0.006	2.026
2006	0.085	0.042	0.074	0.236	0.120	0.064	0.226	0.090	0.181	0.046	0.133	0.183	0.097	0.053	0.250	0.561	0.008	2.447

5. SIMULATION MODEL ANALYSIS

In order to study the water supply and water demand relationship for existing as well as future conditions and to determine the maximum potential for water resources development in the basin, the HEC-3 "Reservoir System Analysis for Conservation" developed by the Hydrologic Engineering Center of the US Army Crops of Engineers in 1981 was used (HEC, 1981). The model simulates the behavior of a water resources system for such conservation purposes as water supply, navigation, recreation, low-flow augmentation and hydroelectric power. HEC-3 model is popular in Thailand at present.

For the analysis purpose, the whole basin in the study area was conceptualized by dividing into three independent main sub-basins (viz. Nam Mae Pai, Nam Mae Yuam, Mae Nam Moei) and a combined sub-basin consisting of two smaller independent sub-basins (viz. Huai Mae Tae Luang and Nam Mae Ngae). HEC-3 was used to run each model configuration for the three water demand scenarios (i.e. existing case - 1993; short term case - 1996; and long term case - 2006) and 27 years of streamflow data reconstituted using the HEC-4 model. The simulation model also considered such details as the minimum flow constraints, return flow components, proposed project characteristics, and diversion requirements at various specified locations. The sub-models were conceptualized and run individually such that, when combined, they yielded the water balance for the whole study area.

For the long-term development by the year 2006, Table 4 gives the summary of water balance for the whole basin. The analysis of long-term situation in 2006 corresponds nearly to full development of the basin at the end of the Ninth National Economic and Social Development Plan (2002-2006). In order to investigate the conservative case, a second long-term scenario was also studied through the HEC-3 model using the presently existing efficiency values. There are some dams/reservoirs and interbasin water diversion projects proposed in the study area which will be realized only during the full development stage of the basin beyond the year 2006. One exception is Mae Lamao-Bhumibol Interbasin Diversion Project in Moei sub-basin, which has been already investigated at the feasibility study level by Newjec et.al.(1993) for EGAT. This project, found to be feasible and anticipated to be constructed during 1998-2002, is considered in the analysis.

For the long-term case, as in the existing and short term cases, some shortages - particularly in meeting the required minimum flows - occur in sub-basins 2, 5, 10 and 16. For the basin as a whole, about 7.2% and 0.6% of the average annual inflow of 8565.67 MCM are diverted for irrigation and domestic/industrial water supply. About 6.2% is actually consumed while 89.8% flows out of the basin. The shortages in irrigation and low flow maintenance are only 2.6% and 3.8% but the shortage in interbasin transfer is almost 57.0%. When irrigation efficiencies are assumed to be the same as existing, the results are similar but are affected by almost two-fold increase in irrigation demands. For the whole basin, about 12.9%, and 0.6% of the average annual inflow are diverted for irrigation and domestic/industrial water supply respectively. The consumptive use and outflow account for about 11.7% and 88.3%. In this case also, the shortages in irrigation and required minimum flows are 9.9% and 5.6% but the shortage in interbasin diversion is 61.9%. The shortages occur mainly during the dry season. Relatively high interbasin diversion shortage is due to the fact that the diversion requirement is based upon the proposed tunnel capacity of 25 m^3/sec.

Table 4 : Summary of Water Balance Study (MCM) : Long Term (2006)

Description	Nam Mae Pai Sub basin Total	Nam Mae Yuam Sub basin Total	Mae Nam Moei Sub basin Total	Independent Sub basin Total	Whole Basin
Inflow	2406.63	2769.90	2412.74	976.40	8565.67
Demand	184.17	153.52	1135.87	4.98	1478.54
(a) Irrigation	164.30	143.10	325.52	1.92	634.84
(b) Others	19.87	10.42	21.95	3.06	55.30
(c) Trans Basin	–	–	788.40	–	788.40
Diversion	176.96	151.00	680.92	4.98	1013.86
(a) Irrigation	157.09	140.58	319.72	1.92	619.31
(b) Others	19.87	10.42	21.95	3.06	55.30
(c) Trans Basin	–	–	339.25	–	339.25
Shortage	8.02	2.52	454.95	0.00	465.49
(a) Irrigation	8.02	2.52	5.80	0.00	16.34
(b) Others	0.00	0.00	0.00	0.00	0.00
(c) Trans Basin	–	–	449.15	–	449.15
Req. Minimum Flow	334.28	310.44	268.46	114.75	1027.93
Actual Min. Flow	328.43	309.26	237.24	114.27	989.20
Shortage (Min. Flow)	5.85	1.18	31.22	0.48	38.73
Return Flow	25.70	12.91	102.14	0.75	141.49
Consumptive Use	151.26	138.09	239.53	4.23	533.12
Out Flow	2255.37	2631.81	1833.96	972.17	7693.30

6. SUMMARY AND CONCLUSIONS

An assessment of water resources as well as water demands in the Salawin river basin in Thailand within the context of national level water resources planning has been presented. The paper only considers the portion of the international basin lying within the territory of Thailand. The study area covers about 17,920 sq.km. of mountainous and relatively fragile watershed in the remote northwestern part of the country. It has been divided into 18 sub-basins for the purpose of analysis.

Hydrological analyses have shown that the monsoon influenced rainfall and surface water resources situations are fairly good, though with a distinct dry season. Ground water resources is limited. Quality of both surface and ground water is good. Water demands for various uses such as irrigation, domestic and industrial water supply, livestock and tourism, hydropower, and environmental control during the planning period (1993-2006) are relatively low. Various water supply versus water demand scenarios analyzed by using the HEC-3 river basin simulation model have indicated that the water supply and demand situation in the Salawin river basin appears to be favorable in the foreseeable future until the year 2006. In fact, the situation looks very good even beyond 2006 for the full potential development of the basin. Hence, the proposals to divert water from the Salawin river basin to the adjoining basins in Thailand, where there are acute water shortage problems, appear to be reasonable provided necessary attention is paid to the socio-economic and environmental impacts of such interbasin transfer projects.

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Techno-political Decision-making for Water Resources Development: The Jordan River Watershed

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ABSTRACT

Discussions on water resources development generally focus on a variety of technical options, often without considering the potential political repercussions of each option. This paper incorporates both technical and political considerations in a 'techno-political' decision-making framework. Water resources development alternatives are then examined to evaluate their respective priorities for development in Israel, Palestine, and Jordan, which are the major riparians of the Jordan River. Particular account is taken of the Middle East peace negotiations, and consequent political changes. Each proposal is designed to provide incentives for sharing resources and benefits among the riparian states.

ABRÉGÉ

Les débats sur le développement des ressources hydrologiques se concentrent généralement sur une variétè d'options techniques qui souvent ne prennent pas en considération les répercussions politiques possibles. Cette communication incorpore les considérations techniques et politiques dans un [decision-making?] cadre "techno-politiques." Alternatives pour le développement des ressources hydrologiques sont étudiées afin d'évaluer les priorités de développement en Israël, en Palestine, et en Jordanie, c'est-à-dire les principaux pays riverains du Jourdain. Aussi, cette communication tient particulièrement compte non seulement des négotiations pour la paix au Moyen Orient, mais aussi des suites des changements politiques. Chaque proposition est conçue pour fournir des motifs qui encourageraient les pays riverains à partager ces ressources et des lors à en bénéficier aussi.

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1. INTRODUCTION[3]

The accord between Israel and the Palestine Liberation Organization (PLO) in Oslo on 13th September 1993, produced a Declaration of Principles which included proposals for an inter-state regional economic development plan (Israel/PLO, 1993). Regional economic development was conceived as a key element to sustaining the peace process in the region (see Figure 1). Similarly, regional watershed development was emphasized in an agenda for cooperation, signed between Jordan and Israel in June, 1994.

This paper examines regional water projects, considering both technical and political aspects of viability. Our focus is "techno-political" considerations of non-conventional water-energy development alternatives for each of the two inter-state regions -- the Dead Sea and Aqaba. Our considerations are in the context of sharing resources and benefits, taking into account the next possible multi-lateral peace agreement among Israel, Palestine,[4] and Jordan. Syria, Lebanon, Egypt and Saudi Arabia could also share resources and benefits from some of these schemes.

2. WATER AND ENERGY AS KEY ISSUES FOR REGIONAL DEVELOPMENT

2.1 Hydro-Political Positions[5]

Before investigating specific projects, the major hydropolitical issues facing each political entity are examined:

Israeli water issues

Sustainable yield of renewable fresh waters in Israel is approximately 1,500 x 10^6 m^3 per annum. Israel had already exceeded this level by the early 1970s, and had to cut 29% from its national water budget from 1,987 x 10^6 m^3 in 1987 to 1,420 x 10^6 m^3 in 1991 due to severe drought. Israel accomplished this without losing net agricultural product or economic growth. Overall water savings in the agricultural sector was 40% during the same period -- from 1,434 x 10^6 m^3 in 1986 to 875 x 10^6 m^3 in 1991.

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4 We use the term, "Palestine" to refer to the West Bank and Gaza, with no pre-disposition to the eventual outcome of the Middle East peace talks.

5 For more in-depth discussions of both the hydro-political positions and the technical and policy options available to the riparians of the Jordan basin, the reader is referred to Murakami (1994) and Wolf (1994).

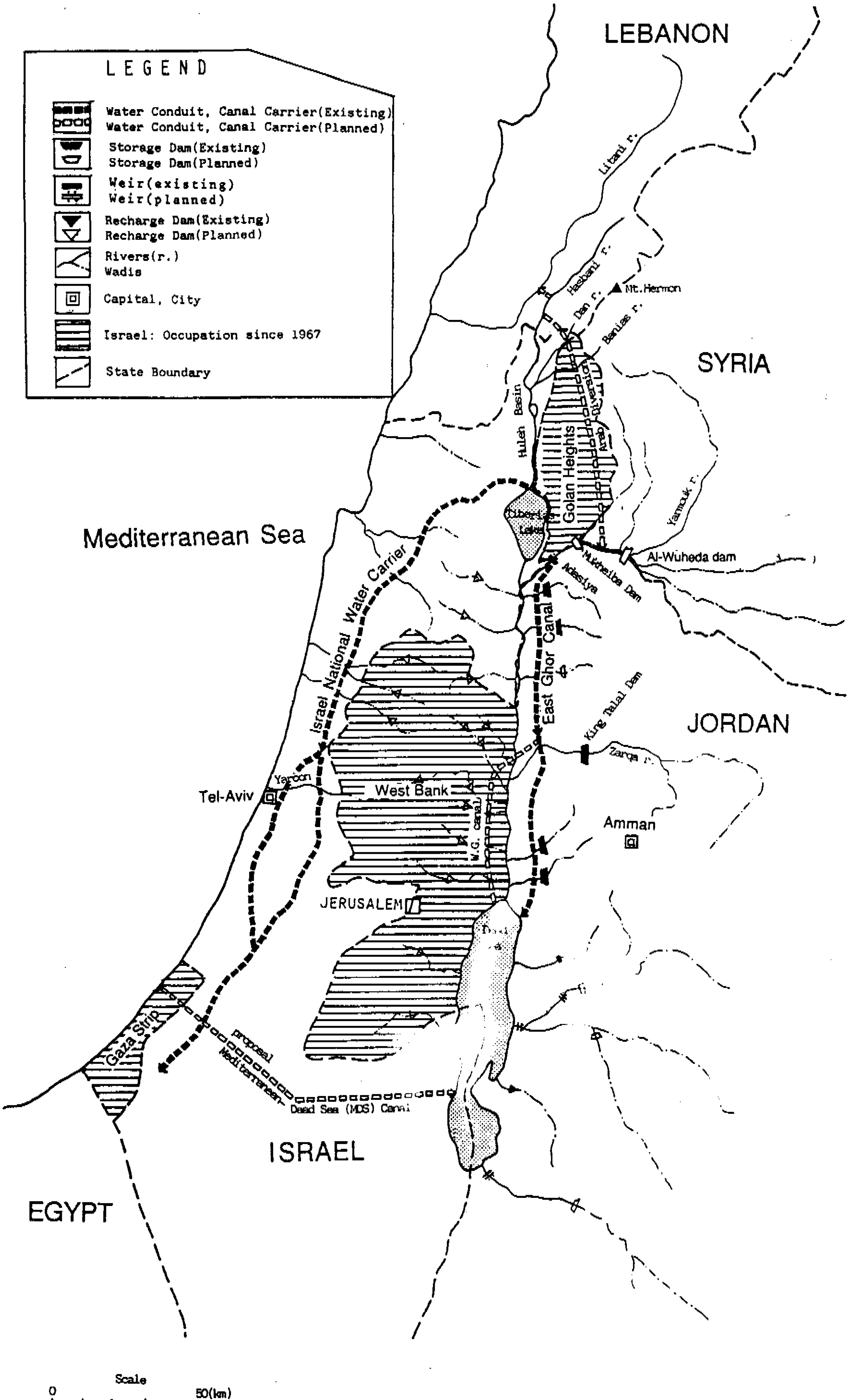

Fig.1 Jordan River System and Water Resources Development Project

Palestinian water issues

Israel took control of the West Bank in 1967, including the recharge areas for aquifers which flow west and north-west into Israel (at about 320 x 10^6 m^3/yr. and 140 x 10^6 m^3/yr. respectively) and east to the Jordan Valley (about 125 x 10^6 m^3/yr.). The entire renewable recharge of these first two aquifers is already being exploited and the third is close to being depleted as well. Total consumption within the West Bank is 35 x 10^6 m^3/yr., mostly from wells, for Israeli settlements; and 115 x 10^6 m^3/yr., from wells and cisterns, for Palestinians. Israel is dependent on the West Bank for a total of some 430 x 10^6 m^3 per annum of its water supply out of a total 1,420 x 10^6 m^3, which accounts for 30% of the annual water potential. Because any overdraft would result in saltwater intrusion along Israel's coastal plain, or eventually even into the mountain aquifers, Palestinian water usage has been severely limited by the Israeli authorities. Palestinians, on the other hand, have claimed first rights to all of the ground- and surface-water which originates on the West Bank, and have objected to Israeli controls. Palestinians were also to receive 70-170 x 10^6 m^3/yr. from the Jordanian share of the Johnston negotiations of 1953-55.

Gaza is probably the most desperate political entity in the region hydrographically. Completely dependent on the 60 x 10^6 m^3/yr. of annual groundwater recharge, Gazans currently use approximately 95 x 10^6 m^3/yr. The difference between annual supply and use is made up by overpumping in the shallow coastal aquifer, resulting in dangerous salt-water intrusion of existing wells and ever-decreasing per capita water availability, which is already the lowest in the region.

Jordanian water issues

Jordan will have exhausted its renewable fresh water resources of 870 x 10^6 m^3/yr very soon. Only two major tributaries of the Jordan river system have not been fully developed -- Wadi Mujib, with an annual flow of 78 x 10^6 m^3, and the Yarmuk, with annual flood flows of 168 x 10^6 m^3 (World Bank 1988). Wadi Mujib, which is the third largest tributary flowing into the Dead Sea, has no inter-state riparian complications as does the Yarmuk. One current question on sustainable water development in Jordan is whether it can afford to continue to develop fossil groundwater in deep sandstone aquifers, like the Disi, and if so then "for what?" and "for how long?".

2.2 Energy-Water Issues

Energy issues relating to water resources are critical matters in the development of non-oil producing countries like Israel, Palestine, and Jordan. These countries are the major riparians of the Jordan river system, and all have increasing demand for desalination and the reuse of treated wastewater, both of which consume substantial amounts of energy.

The current energy sources in the region are heavily dependent on crude oil. Israel, for example, consumes 2.5 million tons of oil and 2.3 million tons of oil-equivalent of coal for the generation of electricity. The annual production of electricity amounted to 20.9 billion kWh at an installed capacity of 5,835 MW in 1991 (State of Israel 1992). Because of its level of development, Israel is investigating the option of replacing its steam power generating system with nuclear by stages into the 21st century. A significant deficit in peak power supply has been a long standing problem, while substantial off-peak electricity is being wasted. Although international networking of the electric supply is being discussed between adjoining states including Egypt, no alternatives have been suggested other than building a new pumped-storage unit and/or gas turbine generating units.

A developed country's energy needs are closely related to its water supply, which can consume substantial electricity to move water from its sources to where it will be used. Pumping demand in Israel, for example, amounted to 1,528 kVA in 1991 (State of Israel 1992), or 30% of total expenditures on water supply by Mekorot, the national water development company. Taking into account recent advances in desalination, Israel is planning to introduce large-scale seawater desalination by the year 2000. Although this is likely to be dependent on low energy types of reverse osmosis, the energy cost will still be 50-60% of the total. Consequently the potential use of off-peak electricity will be a key element in minimizing the cost of water management and operation.

3. TECHNO-POLITICAL DECISION-MAKING

All of the technical and policy options available to any watershed reaching the limits of its water supplies are listed in Figure 2. Once the technical and policy options are known, the next, and probably the most crucial, step is to develop a method for evaluating the options against each other; that is, to create a hierarchy of viability. Many disciplines provide their own version of viability. Where an engineer might ask, "Can it be done?"; an economist might add, "At what cost?" A political analyst could suggest, "Is it politically feasible?"; and anyone environmentally aware might counter, "Should it be done at all?"

One problem with these varied standards of viability is that they often measure at cross-purposes, arriving at differing or even contradictory conclusions. Dinar and Wolf (1991), for example, evaluate a potential Nile to Jordan basin water transfer, both in terms of economic and political viability. Their findings using each standard were in diametric opposition to each other: whereas an economic analysis suggested greater payoffs for larger coalitions of states cooperating, a political investigation showed that the likelihood of such coalitions actually forming decreased as the size of the coalition increased, and that the most likely action was no cooperation whatsoever.

WATER MANAGEMENT OPTIONS TO INCREASE SUPPLY OR DECREASE DEMAND

UNILATERAL OPTIONS

DEMAND

- Population control.
- Rationing.
- Public awareness.
- Allow price of water to reflect true costs (including national water markets).
- Efficient agriculture, including:
    - Drip irrigation
    - Greenhouse technology
    - Genetic engineering for drought and salinity resistance

SUPPLY

- Wastewater reclamation.
- Increase catchment and storage (including artificial groundwater recharge).
- Cloud seeding.
- Desalination.
- Fossil aquifer development.

COOPERATIVE OPTIONS

- Shared information and technology.
- International water markets to increase distributive efficiency.
- Interbasin water transfers.
- Joint regional planning.

Fig. 2

What we suggest here is a unified approach to overall viability which incorporates established measures for technical, environmental, economic, and political viability. Technical viability measures the physical parameters of a system or proposal -- how much water might be produced? what is the quality? how reliable is the source? These physical parameters have well-defined quantitative values which might be used as indicators of viability. Quantity might be measured as volume of water produced by a project within a year, for example. Likewise, quality might be evaluated in parts per million of total salinity or particular pollutants, and a value for flux in a natural system or down-time in a technological project might be an indicator for reliability. Relative environmental degradation can also be evaluated quantitatively, if impact assessments are performed uniformly between projects.

Economic viability has two aspects -- financial, which measures the chances of acquiring financing for a project (often, but not always related to the amount of capital required), and efficiency. For relative water projects, one might use the results of a benefit/cost analysis and use the resulting net present value of benefits as a measure or, more directly, the cost per unit water which would result from each project. An important economic point is that costs are not fixed over time. A 'resource depletion curve' for any project would show at what rate the utility, or value, of a unit of water would begin to drop, and consequently, what the most efficient rate of development would be.

The most tenuous measure is political viability. To incorporate this important parameter in an integrated model, one must use a relative scale for a value which is difficult to quantify. While we recognize the general lack of enthusiasm for quantitative political analysis for its necessarily subjective nature (see Ascher 1989, for a good critique), we recommend the inclusion of results of a process such as the PRINCE Political Accounting System. Coplin and O'Leary (1976) describe the method of incorporating each player's "position," "power," and "salience," for any of a number of policy options to arrive at a relative ranking of political viability. In Coplin and O'Leary (1983), they extend the process to provide an absolute measure of the likelihood of a policy action taking place.

Two other qualitative measures might be used for political viability. For projects within a country, how well a proposal "fits" with national goals might be evaluated. Population control, for example, which might be successful in western Europe or the United States, runs counter to both Israeli and Palestinian interests in numerical superiority. International projects might be determined in terms of relative measures for "equity" of project costs and water distribution, and "control" by each political entity of its own major water sources.

If the resources are available to perform a detailed feasibility study, the results can be described quantitatively. Listed below are the proposed measure of viability, followed by the possible quantitative standards which might be used:

-- Engineering

- quantity (eg. x 10^6 m^3/yr.);
- quality (eg. ppm salinity or pollutants);
- reliability of source (eg. std. deviation of flux);

-- Environmental

- environmental impact (eg. detail of potential damage);

-- Economic

- financial (capital necessary to finance project);
- efficiency; (cost per unit of water, or net present value of benefits);

-- Political

- political probability from PRINCE model; or equity of project cost and water distribution and control of source by each entity.

More often than not, the detailed data necessary for a quantitative evaluation are not available. In that case, two options exist. The first is to substitute qualitative values: +, 0, -, for example, representing good, neutral, or poor; adequate for a preliminary analysis. We can then evaluate any possible option qualitatively with each measure of viability. By examining the results, we should get a sense of which options are more viable than others, and why. It should be remembered that these results are for a particular geographic location, and for a single point in time.

Although a column is provided for a measure for "overall viability," it is recommended that, if the column is used at all, it is used with tremendous caution. First, each measure does not necessarily have equal weight, and each was arrived at with both some subjectivity and some uncertainty. Adding or multiplying across would therefore only compound and accumulate error.

Also, by leaving the measures separate, one acquires a greater sense of why options are viable, and where emphasis can be placed for the future in order to help boost viability. Public awareness, for example, has been shown to be a very cost-effective method of saving water, but the total amount which can be saved is rather small as compared to the total water budget. In contrast, unlimited water can be made available through desalination but at a relatively higher cost. The latter might change with technologic breakthroughs, but the former is likely to remain fairly constant over time.

The second option in the absence of data necessary for a quantitative assessment is to substitute iterations of "expert opinions," first described by Gordon and Helmer (1964) as the Delphi method.[6] Experts familiar with the technical and political landscapes of a particular watershed might be asked to rank available options as to their viabilities on a consistent scale. The viability measures themselves should also be weighted as to their relative importance for that particular watershed during a particular time frame. A variation on the weighting process is first described in detail in Kepner and Tregoe (1965).

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6 See Linstone and Turoff (1975) for a good summary of the strengths, weaknesses, and applications of the Delphi method; and Needham and de Loë (1990) for its applicability to water resources planning.

It should be emphasized that this evaluation process should be iterative -- repeated often to allow for the constant changes of so many of the parameters over space and time. Changes which can affect viability include:

--Technical and Environmental:

- Fluctuations in seasonal and annual water supply, as well as long-term changes due to global warming.
- Changes in water quality.
- Technical breakthroughs.
- Relative infrastructure for each party in
  - research and development,
  - storage and delivery,
- Changes in understanding of physical system.

--Economic:

- Changing priorities for funding agencies.
- Movement along the resource depletion curve.
- Expense for water resources development.
- Changes in efficiency of water use.

--Political:

- Power relationships
  - riparian position,
  - military,
  - legal (eg. clarity of water rights).
  - form and stability of government.
- Level of hostility.

The evaluation process should also allow for interaction, with on-going feedback between the disciplines, to reflect real-world influences. For example, a project with extremely positive economic results might help overcome political reluctance to enter into cooperation. Likewise, political constraints can effectively veto a project which has been judged worthwhile in terms of its technical and economic value.

4. SPECIFIC OPTIONS AND "TECHNO-POLITICAL" VIABILITIES

4.1 Perspectives of Non-conventional Water Development Alternatives

Conventional alternatives, which include surface water and groundwater development, have the highest priority in water resources planning where there are still renewable fresh waters to be developed and intricate inter-state riparian questions do not result. This ideal situation does not exist in most countries of the Middle East. After exploiting all of the renewable fresh water resources within their national boundaries, Israel, Palestine, and Jordan will have no choice except to develop transboundary waters and/or non-conventional waters. Water conservation is an important and essential issue in water management, and outside sources should not be considered until sources within the basin are being used at where:

U : maximum velocity of water in pipes.

e) water may only be obtained from supply points where intake structures are located, and the quantities withdrawn must not exceed the amounts available at these points; therefore, one shall write (water supply constraints):

$$q^N_j \leq q_{j\,\max}\, x_j \ , \quad \forall j \in J^S .$$

f) water obtained through a given pump must not exceed the capacity of the pump; in mathematical terms we can write (pump power constraints):

$$\sum_{p \in P} p\, y_{jp} \geq \frac{\gamma\, h^N_j\, q^N_j}{\eta}, \quad \forall j \in J$$

with

$$\sum_{p \in P} y_{jp} \leq 1 , \quad \forall j \in J$$

and

$$\sum_{p \in P} p\, y_{kp} \geq \frac{\gamma\, h^L_k\, |q^L_k|}{\eta}, \quad \forall k \in K$$

with

$$\sum_{p \in P} y_{kp} \leq 1 , \quad \forall k \in K$$

where:

γ: specific weight;

η : pump efficacity.

g) total costs to be minimized can be expressed (assuming variable intake costs to be independent of the supply point where water is withdrawn) as:

$$\min C = \sum_{j \in J} C_{\text{intake}_j}\, x_j + \sum_{j \in J} \sum_{p \in P} C_{\text{pump}_p}\, y_{jp} + \sum_{k \in K} \sum_{p \in P} C_{\text{pump}_p}\, y_{kp} +$$

$$+ \sum_{k \in K} \sum_{d \in D} C_{\text{pipe}_d}\, z_{kd} + \sum_{j \in J} C_{\text{energy}}\, h^N_j\, q^N_j + \sum_{k \in K} C_{\text{energy}}\, h^L_k\, |q^L_k| .$$

The complete optimization model is therefore:

$$\min C = \sum_{j \in J} C_{\text{intake}_j} x_j + \sum_{j \in J} \sum_{p \in P} C_{\text{pump}_p} y_{jp} + \sum_{k \in K} \sum_{p \in P} C_{\text{pump}_p} y_{kp} +$$

$$+ \sum_{k \in K} \sum_{d \in D} C_{\text{pipe}_d} z_{kd} + \sum_{j \in J} C_{\text{energy}} h^N_j q^N_j + \sum_{k \in K} C_{\text{energy}} h^L_k \left| q^L_k \right|$$

s.t.

$$\sum_{k \in K_j^-} q^L_k = \sum_{k \in K_j^+} q^L_k + q^N_j , \forall j \in J^S$$

$$\sum_{k \in K_j^-} q^L_k + Q_j = \sum_{k \in K_j^+} q^L_k , \forall j \in J^D$$

$$\sum_{k \in K_j^-} q^L_k = \sum_{k \in K_j^+} q^L_k , \forall j \in J^A$$

$$h^N_j = h^N_{j_{ref}} + \sum_{j' \in J} \alpha_{j'j} q^N_{j'} , \forall j \in J$$

$$h^L_k = \max \left[0, \Delta H_{g_k} + \phi \frac{{q^L_k}^2}{\sum_{d \in D} d^{\frac{16}{3}} z_{kd}} L_k \right], \forall k \in K$$

$$\sum_{d \in D} z_{kd} \leq 1, \forall k \in K$$

$$q^L_k \leq \frac{\pi}{4} U \sum_{d \in D} d^2 z_{kd} , \forall k \in K$$

$$q^N_j \leq q_{j_{\max}} x_j , \forall j \in J^S$$

$$\sum_{p \in P} p\, y_{jp} \geq \frac{\gamma h^N_j q^N_j}{\eta}, \forall j \in J$$

$$\sum_{p \in P} y_{jp} \leq 1, \forall j \in J$$

$$\sum_{p \in P} p\, y_{kp} \geq \frac{\gamma h^L_k \left| q^L_k \right|}{\eta}, \forall k \in K$$

$$\sum_{p \in P} y_{kp} \leq 1, \forall k \in K$$

$$x_j, y_{jp}, y_{kp}, z_{kd} \in \{0,1\}, \forall j \in J, k \in K, p \in P, d \in D$$

$$h^N_j, h^L_k, q^N_k \in \mathbf{R_0}^+, \forall j \in J, k \in K$$

$$q^L_k \in R, \forall k \in K$$

3. SOLUTION ALGORITHM

The model formulated in the preceding section is a very realistic and complex one, and it is quite doubtful that it can be solved by the methods reported in the literature dealing with branched water supply design problems.

Some years ago, the very idea of solving such a model to optimality (and even to near-optimality) would be unreasonable, but some recent random search optimisation algorithms, like annealing algorithms, allow its resolution to be faced with some hope of success(1).

The first example of an annealing algorithm is due to Metropolis, Rosenbluth, Rosenbluth, Teller & Teller, 1953 and has been developed for a thermodynamical problem which was not, at least explicitly, an optimization problem (it was a matter of simulating the evolution of a spin glass towards the state of minimum energy as the temperature is progressively decreased).

Quite recently, some authors, namely Kirkpatrick, Gelatt Jr & Vecchi, 1983 and Cerny, 1985 had the innovative idea of applying the principles of the Metropolis algorithm to a well-known difficult combinatorial problem, the travelling salesman problem, and very good solutions were obtained.

Following them, other applications of the same principles have been done, usually with rather interesting results(2); one of these applications has been done by one of the authors of this paper on a very complex educational facilities planning problem (see Antunes, 1994).

The basic idea behind annealing algorithms can be described as follows.

Suppose that the cost of the current configuration, s, of a given system (in the present case, a configuration is defined by the pumps and the pipes that integrate the network, and by the supplies that are carried through the network to demand centers) is $c(s)$.

It can be shown (see Aarts and Korts, 1989) that if the transition of that configuration to a candidate (neighbouring) configuration, s', of cost $c(s')$ (choosen at random in such a way that all possible system configurations will remain permanently accessible, directly or indirectly) is made with probability $p = \min\{1, \exp(\Delta c)/t\}$, with $\Delta c = c(s) - c(s')$ and t being a parameter, and if t is decreased in an adequate manner, the system will converge to the global least cost configuration as the number of transitions attempts increases(3).

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1 Other random search techniques are usually referred as tabu search, genetic algorithms and neural networks (see Reeves, 1993).

2 For a survey of annealing algorithms applications see Eglese, 1990.

3 The existence of convergence does not means rapid convergence (see, Lundy & Mees, 1986).

This property of convergenge to a global optimum cost configuration stems from the fact that transitions from high to low cost configurations are not automatically excluded, they will take place or not depending on the difference between costs and the level of temperature; initially, even very negative (counter-optimum) transitions will be accepted; with temperature falling, their acceptance will become more and more rare.

In general term, an annealing algorithm will therefore include the following steps:

1. choose s_1 {s_1 is the initial configuration}
2. choose t_1 {t_1 is the initial temperature}
3. choose t_f {t_f is the stopping temperature}
4. $j \leftarrow 0$
5. **repeat**
   6. $j \leftarrow j+1$
   7. choose at random $s'_j \in N(s_j)$ {$N(s_j)$ is the candidate set of s_j }
   8. choose at random $p \in [0, 1]$;
   9. **if** $p \geq \min\left\{1, \exp\left(\frac{c(s_j) - c(s'_j)}{t_j}\right)\right\}$

      then $s_{j+1} \leftarrow s'_j$

      else $s_{j+1} \leftarrow s_j$
   10. choose $t_{j+1} \leq t_j$

   until $t_{j+1} \leq t_f$
11. **end**

Any adaptation of this general algorithm to a specific problem will involve decisions concerning two crucial issues: the set of configurations that are directly accessible from the current configuration (the candidate set); and the value that the temperature will take in successive transition attempts (the cooling schedule).

The first issue is particularly important in the problem treated in this paper.

Up to this point of research, the best candidate set is obtained by perturbing the current configuration as follows:

a) 8 out of 10 times an initial change is introduced in the flow circulating in one of the network pipes, selected at random; in the remaining 2, it is introduced in the flow pumped at one of the supply nodes, also selected at random;

b) 8 out of 10 times the initial change in flows amounts to a fraction of the total pipe or node flow, defined at random in terms of a flow unit previously specified; in the remaining 2, it consists in the elimination of the flow circulating in the pipe or pumped through the node, and thus to the elimination of the pipe or the pump;

c) in order to restore equilibrium at unbalanced nodes, compensating changes are introduced at some pipes and at some nodes, choosen at random.

The method used for this purpose is explained hereafter, through the example illustrated in Figure 2, (a) and (b).

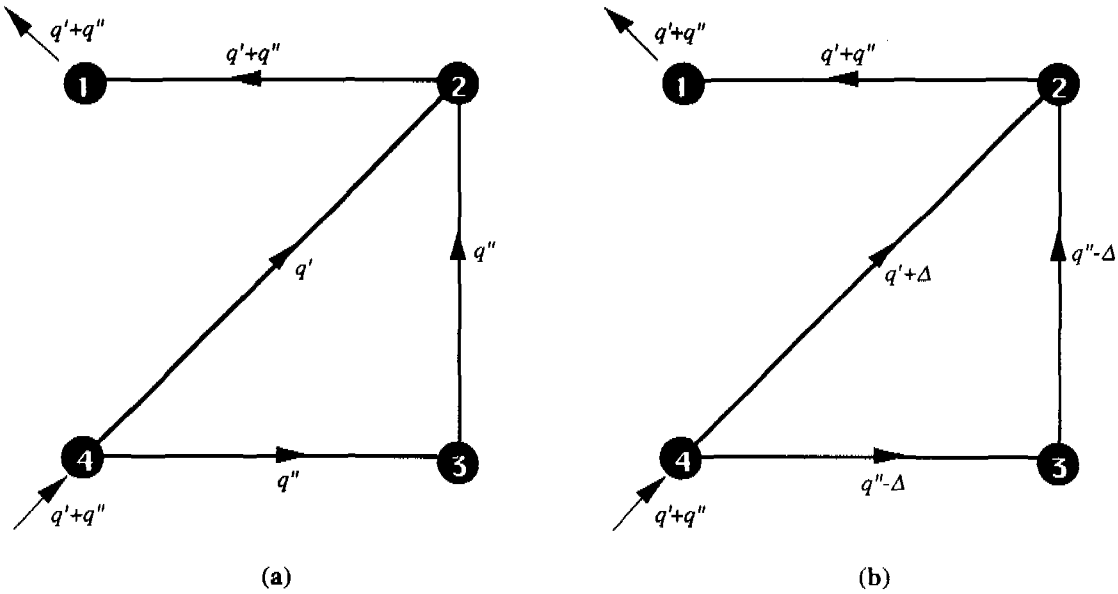

Figure 2: Restoring Node Equilibrium

Figure 2(a) shows the current flow configuration of the network, with every node in equilibrium.

Suppose that the initial change, randomly selected, occurs at pipe 24 (pipe having nodes 2 and 4 as extreme nodes), and that it consists of an increase of Δ units of flow; this causes the flow at pipe 24 to become $q'+\Delta$ and a disequilibrium at node 2 (and also, of course, at node 4).

To restore equilibrium at node 2 a decrease of Δ units of flow must be induced at one of the pipes converging to it; admit that pipe 23 was randomly selected; this causes the flow at pipe 24 to become $q''+\Delta$ and a disequilibrium at node 3.

To restore equilibrium at node 3 a decrease of Δ units of flow must be induced at pipe 34 (in this case one needs not to select a pipe, because there is only two pipes converging to the pipe; otherwise one should proceed as for node 2); this decrease automatically restored equilibrium at node 4, and at the network as a whole.

Figure 2(b) shows the candidate flow configuration of the network, resulting from the described changes.

The second issue, concerning the cooling schedule, is less problem specific; among the available alternatives the one proposed in Johnson & al., 1991 is quite appealing and has been selected.

Their schedule is a discrete schedule defined in terms of four parameters:

— a :the probability of accepting a transition from the starting configuration to a candidate configuration whose costs are superior to those of the starting configuration in a given percentage (this parameter is used to define the initial temperature of the annealing process);

— n_1: the number of algorithm iterations that will be performed without an improvement of the optimum or the average cost of current configurations before decreasing temperature;

— r : the rate at which the temperature is decreased, whenever a temperature decrease should occur;

— n_2: the number of temperature decreases that will be performed without an improvement of the optimum or the average cost of current configurations before stopping the algorithm.

The way these parameters are used together is illustrated in Figure 3.

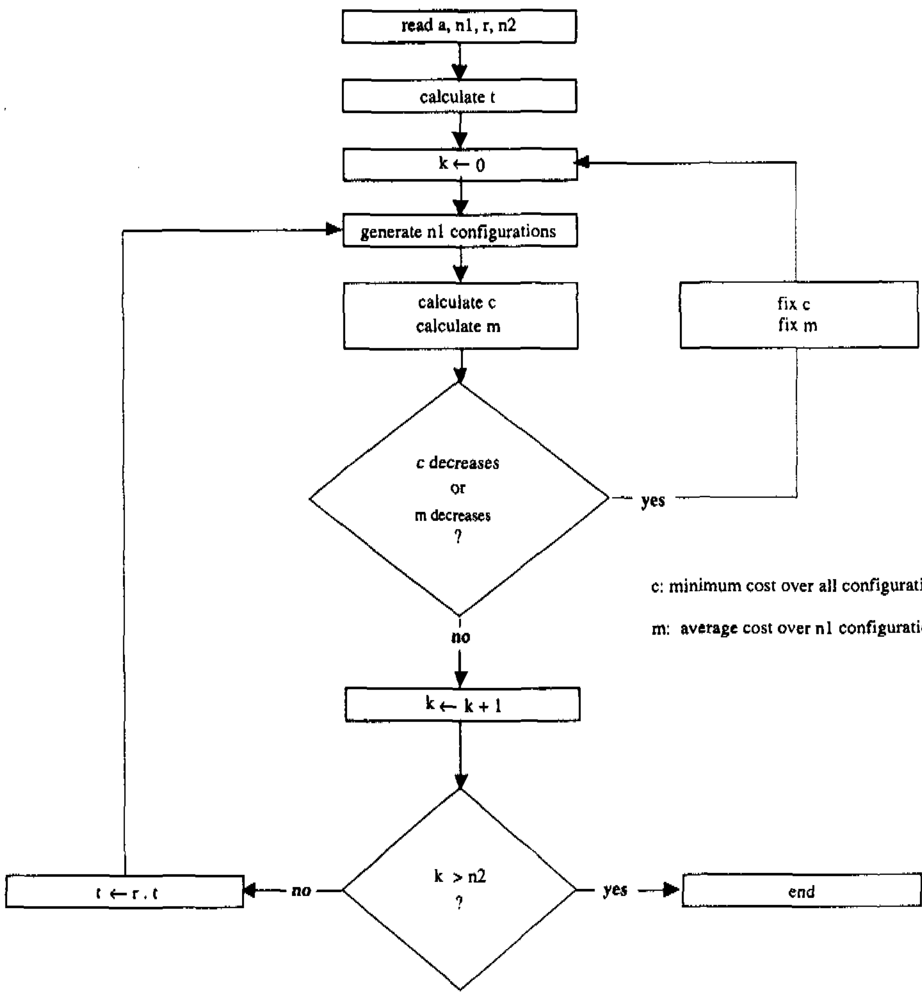

Figure 3: Cooling Schedule Parameters

4. PRELIMINARY RESULTS

At this stage the only results available concern three small test problems that are being used for the purpose of developing and perfecting the computer code corresponding to the algorithm presented in the preceding section.

The first problem to be considered (from now on named reference problem) was built around the geography depicted in Figure 4 (of course, a very simple situation was retained, in order to facilitate the manual confirmation of results); it can be described as follows: there are two demand centers to be supplied, located at 1 and 2; there are three supply points that may be used, located in 3, 4 and 5, all withdrawing water from the same aquifer; demand requirements and supply conditions are included in Table 1; what structures should be installed in order to meet those requirements at minimum cost ?.

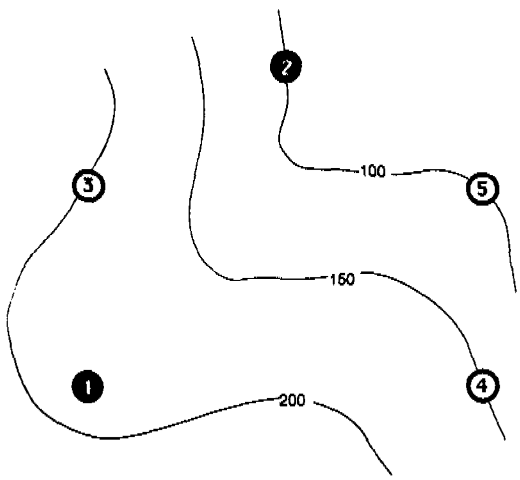

Figure 4: Reference Problem Geography

Variables	Centre 1	Centre 2	Point 3	Point 4	Point 5
X - Coordinate (km)	0	5	0	10	10
Y - Coordinate (km)	0	8	5	0	5
Water Requirements (l/s)	350	230	-	-	-
Head Requirements (m)	180	100	-	-	-
Water Availability (l/s)			600	600	600
Water Depth (m)			10	10	10
Soil Level (m)			200	150	100
Influence Coefficients (m/m3/s)					
Point 3	-	-	3.0	0.5	0.3
Point 4	-	-	0.5	8.0	3.5
Point 5	-	-	0.3	3.5	10.0

Table 1: Reference Problem Data

The solution to this problem is rather obvious (pump 580 l/s of water at point 3 and send 350 l/s to centre 1 and 230 l/s to centre 2 (for normal pump, pipe and energy costs, like those that has been used in this study); the question is: is the algorithm able to find this solution ? (and, if so, would it still be able to find it when, following some data changes, the solution is made less obvious ?).

In order to use the algorithm presented before, two previous operations must be performed: the definition of a pipe super set containing the optimal pipe set; and the definition of the cooling schedule parameters.

In what concerns the first operation it must be said that it is not as important as it may seems. because even if one takes a large super set containing obviously superflous pipes, the algorithm will automatically discard them rather quickly, and will concentrate on the most promising ones.

The super set choosen for all the test problems is represented in Figure 5; it contains pipes that are clearly useless, like pipe 35 (what for would it serve ?) — but they were left precisely to verify whether the algorithm will be able to discard them from optimal solutions or not.

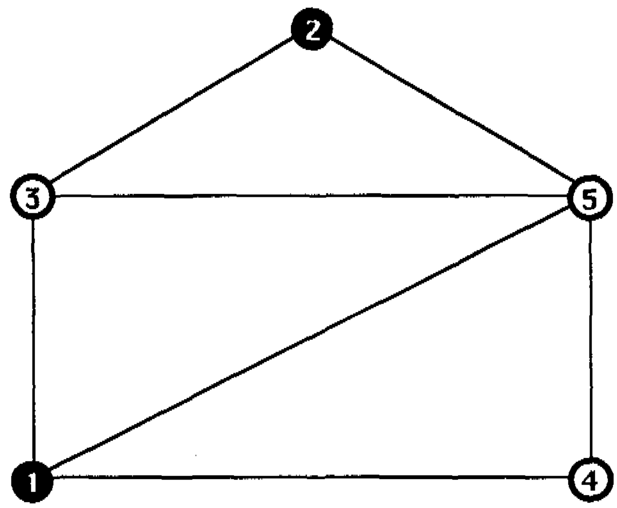

Figure 5: The Pipe Super Set

In what concerns the second operation, after some specific experimentation (based on earlier experimentation with other problems) it was decided to use the following cooling parameters: $a = 0.2$; $n_1 = 4$; $r = 0.2$; $n_2 = 4$.

The results obtained for the reference problem were very promising: in fact, starting from quite illogical initial solutions (though using the highest supply point), the algorithm has been able to find the optimal solution in 10 out of 10 runs, within run times going from 65 to 130 seconds[4] (it should be noted here that, annealing algorithms being random algorithms, their results depend on the random sequences used in generating candidate configurations and in deciding their acceptance).

The second test problem was built from the first one just by lowering the soil level at Point 3 from 200 to 140 m; this complicates the optimum search because, even though the solution is the same, Point 3 is no longer the highest point (in the initial solution, supply was therefore based in Point 4).

Also in this case, the algorithm has performed quite well, and it was able to find the optimal solution in 6 out of 10 runs, within run times going from 116 to 179 seconds; but in 3 of the remaining 4 runs the solution found was distant to the optimal one in more then 10% (but less than 15%).

The third test problem was also built from the first one as well and also by changing the soil level at Point 3, but this time lowering it from 200 to 80 m, creating a much more complicated problem, full of local optima (whose solution, it must be said, it is not known); in 10 runs, 10 different results were obtained, costs being in the worst case almost 15% greater than those corresponding to the better solution (which possibly is the optimal one); and the run times reached, in 2 runs, values superior to 200 seconds.

CONCLUSION

The study reported in this paper is far from ended, and it is not yet possible, at present, to draw any firm conclusions — except that the solution algorithm needs to be considerably ameliorated before it can be used with some confidence in what optimal solutions are concerned; but one must recognize that, even at this stage, it already may help in finding a reasonable solution to a small water supply design problem.

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[4] The computer used for these runs has been a Macintosh Powerbook 230 running at 33 MHz and with no mathematical co-processor, and the algorithm has been coded in Think Pascal.

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THE ROLE OF GIS IN SATISFYING FUTURE NATIONAL WATER DEMANDS

Mona El-Kady[1]

ABSTRACT

No matter where we go today, the conversation sooner or later ends up in a discussion of the potential benefits of using modern technology tools in the very vital issue of sustainable water and land management and mainly to satisfy the future national and global demands.

The use of GIS technology and its integration with models opens a new horizon before us.

This paper will define the role of GIS technology in satisfying future Egyptian water from the experience gained by the direct involvement of several GIS projects and attending a number of GIS conferences and workshops.

More than on study case using GIS will be discussed as well as their benefits from a management perspective. Also, a discussion of the use of GIS technology if it is a necessity or luxury for developing countries.

1. INTRODUCTION

Egypt's agricultural sector is unique, in that :

* It has some of the richest agricultural resources in the world, having a year-round favorable climate, fertile alluvial soils, and its share of high-quality Nile Water supply.

* Over 95% of its production is derived from irrigated land and its irrigation waters originate totally outside its border.

* Water is the most limiting factor for agriculture

* A rapidly increasing population which is placing severe demands on the production of the agricultural lands.

* Millions of acres have been cultivated for several millennia. Land division over time has resulted in seventy percent of farm being one acre or less in size.

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* Dynamic cropping pattern, as farms are composed of smaller fields and plots and intensive crop rotation is practiced with two to three crops grown per year. Some time more than one plant is cultivated in the same time (inter cropping system).

* Its long history developed generations of experienced specialist and experienced farmers and built mountains of information about the Nile River, the irrigation system and the agricultural land.

Today's Conference is addressing the question of how to satisfy future water demand and to be able to do that we better know what is the future water requirements?

Accurate water requirement data for agricultural lands is a key element needed for efficient management of irrigation and drainage systems water needs, irrigation scheduling and drainage requirements are highly dependent on cropping information.

Having the information is not the only task to develop knowledge in the field of water requirement or any other field, more importantly to have accurate, up to date information and the ability to manage this information.

Using modern technology tools is always a venue to a better management.

2. NEW TECHNOLOGY

From the down of recorded history, Egypt has acknowledge its dependence on the Nile, and Egyptian have sought to manage its life giving water as effectively as the day's technology allowed. Those charged with this awesome responsibility have continually soughtout and implemented new technologies whenever practical.

Today's generation of Egyptian irrigation engineers, planners, administrators is no exception. They are currently enlisting information management technology whenever practical and affordable in this never ending task GIS, as one of the tools of the information technologies introduced during the last ten to twenty years and has been used extensively by many governmental and private sectors.

It has been used by the Egyptian MPWWR in several locations two example of its use to support satisfying water demands, are worthwhile to mention.

- The first is GIS to Manage water resources a pilot program funded by the MPWWR-WRC-SRI and the canadian IDRC.

- The second is a "crop inventory GIS" developed by the S&M component of the IMS project funded by USAID.

3. THE ROLE OF GIS

As many other new technology tools, there is no single accepted definition for a geographic information system.

* GIS is nothing more than a software tools designed for managing, analyzing and displaying the contents of a database containing data which has been geocoded; that is, assigned unique identifiers relating selected elements in the database to their physical location on the surface of the earth or space.

* GIS is the integration of computer graphic technology with database management technology so-that the former can be used to query, update and visualize the contents of the latter or the results of an analysis based on data contained in the latter.

* GIS is the ability to tie information to geography and linking the divers functions to its physical location. It enable our ever-smarter computers to tell us where things are and how they are related to each other (use of geographic location as an index).

If we adopt any of the above functional definition of a geographic information system, in general GIS can play any of the following roles:

1. Organizing information and historical relations

    - Information we manage about our water resources is overwhelms our ability to thumb through a file and understand the contents The information is all valuable, can give a specific direction and in some cases contain several generation of calculations, drawings, description etc...
    The significant time saver is organization of previous work and a tool to easy the use and analysis.

The goal in organizing the mountain of paper files is to view the information you need in the most appropriate and simple way for the use you need it for.

With an appropriate GIS a new personnel can quickly view data and learn to make decisions as opposed to researching information buried in several file drawers or in several places.

Moreover, now your system of organization really benefits by making a query on the data lists and being able to high-light " points, lines and polygons" in the graphic environment, GIS is a tool to organize data and information in a way to allow the user to focus on the geographical and data issues with equal ease, it allow ease of viewing information and ease of analysis.

2. Analysing information

GIS has the capabilities to analyses information as GIS Software becomes more even complex it can be integrated with any mathematical, statistical, hydrological models... to develop certain analysis to support decision markers. The geographic search capabilities of a GIS can be used to collect the data needed by the model and the display capability of a GIS to visually report the results of the data analysis as a readily understandable display.

3. Planning and managing resources

GIS can help in planning and managing resources by creating links among isolated database, enhancing the use of information as a strategic resource throughout any organization however, it must be understood that GIS. Like any other information system, is only as good as the information it analyses - " Garbage in, Garbage out" applies.

Information is a resource, and technology is a tool. Information must be planned for, budgeted and controlled. A GIS is a toll that can help managing and enhancing information and activities so that better decisions can be made.

In specific to satisfy water demands in addition to the above general role GIS is used in Egypt for:

1 - Crop inventory - monitoring the dynamic crop pattern of Egypt's agricultural land.
2 - Estimating the size of the total area of land under cultivation and the area occupied by the main crop varieties.
3 - Estimating accurate water requirements.
4 - Developing water and energy budget analysis for certain areas.
5 - Trace flows through irrigation and drainage network from source to origin and integrating with different models creating network analysis to help in the day operation of the network and irrigation scheduling and visualize the results for network irrigation management.

It is not surprising that new uses and applications seems to appear with each implementation.

GIS is being used for planning different resources including water, environmental protection, community development, integrated public safety response, infrastructure management, transportation planning and modeling, assessments, facility siting, vehicle routing, permitting and licensing, election management, real estate management, seismic safety and hazards management.

The following list has been discussed in many conferences as reasons (reported by governments) to use GIS, they can be grouped into two basic categories.. internal and external operations.

* Internal management and productivity gains include the following improvements : Better data management resulting in lower costs and better planning and decision making.
* Establishment of more accurate links between technology tools and expected productivity improvements.
* Expanded capacity for data storage and manipulation.
* Greater responsiveness through rapid, share access to more data and comprehensive information analysis and reporting capabilities.
* Improved regional planning and intergovernmental communication.
* Increased revenues as charges for new products and services, including public access to GIS data for citizens or businesses are implemented.
* Saving in time, money and staff through reduction or elimination of redundant activities and streamlined operations.

External operations involve the improvement of service to citizens and business constituent. GIS technology generally results in :

* Quicker, more accurate and more consistent responses to citizen inquires.
* Increased availability of old and new products services.

4. CROP AND SOIL INVENTORY GIS PROJECT

The size of the total area of land under cultivation, as well as accurate water requirement data for agricultural land is a key element needed for efficient management of irrigation and drainage systems.

Satisfying water needs, irrigation scheduling, drainage requirements, operation & management of the irrigation and drainage systems are highly dependent on the cropping information.

Currently the USAID funded Survey and Mapping (S&M) project is conducting a detailed crop and soil mapping program through interpretation of CIR film coverage of the entire Nile Delta. The end product of this project is digital crop and soil overlays for the Winter 90-91 and summer 91 crop season in a geographic information system (GIS) database. This is the first time that CIR film has been used for crop classification in Egypt. Detailed crop inventory has been achieved with the 1:20,000 scale CIR film transparencies because of the films high spatial resolution (greater than one meter) and unique spectral properties. The minimum mapping unit size on this study is 0.6 feddans. Photo polygons are transferred to 1:10,000 orthophoto base maps which were scan digitized into an ARC/INFO GIS. The crop and soil classification legend of this project is listed in Table 1.

Mapping Unit	Winter Crops	Summer Crops
1	Wheat	Rice
2	Clover	Cotton
3	Fava	Maize
4	Orchards	Orchards
5	Potatoes	Potatoes
6	Sugar Cane	Peanuts
7	Mixed Row Crops	Mixed Row Crops
8	Other Crops	Other Crops
9	Fallow Field	Fallow Field
10	Lands under Reclamation	Lands under Reclamation
11	Stressed Crops*	Stressed Crops*
12	Non Productive Land**	Non Productive Land**

* Areas under stress due to soil salinity, waterlogging or other reasons. This class can also be used as a modifier of crop classes to yield additional classes(e.g. stressed wheat, stressed clover,etc.).

** Absence or non growth of crops in predominantly arable area due to soil salinity, waterlogging or other reasons

Table 1 Survey and Mapping project CIR crop and soil photo legend Egyptian Survey Authority, Geonex International.

CIR aerial photography has an added advantage for crop studies in Egypt as it is ideally studies for interpretation and mapping of stressed crops and non-productive saline areas. In Egypt, stressed crops are commonly associated with soil salinity and water-logging caused by over irrigation and/or inadequate drainage. A secondary goal of the CIR mapping program of the S&M project is to delineate areas where crop growth is being adversely affected by salinity and waterlogging.

A pilot area in El-Beheira governorate has been chosen to illustrate the different capabilities of the GIS. Layers of information about the total cultivated area the different cultivated crops (Winter & Summer), the irrigation network, drainage network, well, irrigation and drainage problems areas has been developed. Time is coming very soon for decision maker in both the ministries of PWWR and agriculture to make use of these information.

5. GIS FOR WATER RESOURCES MANAGEMENT IN EGYPT

During 1991, the Survey Research Institute (SRI) of the Water Research Center, the canadian International Development Research Center (IDRC) and Energy mines and Resources of Canada (EMR) started a GIS project to support water resources management. The overall objective is to develop a GIS for the storage, manipulation and analysis of data that will be applied for water resources management in Egypt.

The specific objectives are as follows :

- Collect and review research concerning GIS and computer modelling procedures.
- Analyze existing water resource problems.
- Select, modify and adapt existing models to address water resource problems.
- Collect thematic data in a pilot area and develop corporate database.
- Develop the applications software and acquire hardware to support the database, software and models.
- Apply and monitor a hypothetical spatial modelling scenario.
- Ascertain the future of the GIS in the context of regional water resources management.

Anticipated Results

It is anticipated that the following results will be achieved :

- A prototype hardware system that will facilitate the modelling of water resources and development and management problems.
- The integration of spatial and point in the solution of water resource problems.
- A comprehensive database of a pilot area.
- The first step towards a regional GIS at a scale of the entire Nile Delta.
- Analysis of the success of the proposed GIS in the context of regional scale studies.
- Development of expertise by the Egyptians in the development and use of GIS.
- Publication in scientific journals and presentation at various conferences.

Real Results

The information of Ben-Magdool area is very well organized in a well designed data base consisting of base mape with coordinates and GPS points.
Layers of information about : Land ownership
Crop pattern
Canals
Wells
Drains
Irrigation features
Drainage
Roads
Soil classification
Land use
Etc.

Also tabular data about social and economic features of the area is sorted. Water requirement model has been linked (FAO model). Water and energy requirement analysis has been developed. Several queries has been answered starting from very simple one such as what is the best soil area to cultivate certain crop, its water requirement and soil condition to complex query to be answered through using different water budgeting sofesticated models.

5.1 Necessity or Luxury

While by no means a panacea for all that ails a developing country - as some may claim - prudently implemented, GIS technology does offer a forward-thinking developing nation the opportunity to better manage its human and material resources for the good of its progeny.

A number of demonstration projects in Egypt have just started to conclusively show that the wholesale implementation of GIS technology could improve the country's ability to manage its land, water and agriculture resources. When viewed a purely technically perspective, it is safe to say that the coordinated application of GIS technology should proceed forthwith throughout a number of Egyptian ministries. There is little doubt in the minds of those associated with these demonstration project that GIS technology is now a technical necessity for Egypt.

Unfortunately, the wholesale implementation of GIS technology will cost millions, if not billions, of Egyptian pounds which are not now available in the government's budget and which are not likely to be available in the foreseeable future. From a political and financial perspective, GIS technology is a luxury and is likely to remain a luxury for years, if not decades, to come. What then is Egypt to do ?

For the present time Egypt must continue to rely on international development organizations such as IDRC,GTZ and USAID to fund the development of its GIS system. Until now these organizations have only been willing to fund small demonstration or pilot projects. If GIS technology is to be implemented on a large scale any time soon within Egypt, a way must be found to convince these and other international development organization that it is in their own, as well as Egypt's best interest that GIS technology move from its current position as a nice to have luxury to a cannot do without tool for national development.

5.2 Lessons Learned

1 The successful implementation of GIS technology depends first and foremost on the availability of sufficient number of individuals interested in learning the technology, and having a suitable educational background and practical experience with related technologies. As few developing countries can meet this condition training phase should not be over looked or underestimated in any project.

2 Any one who has worked in a bureaucracy knows that control of information is the ultimate source of power, government agencies are normally highly bureaucratized highly centralized. Any system that offers to change the traditional approach to information collection, management, and dissemination will immediately be seen as a threat to the entrenched power holders. The task of introducing GIS technology to governments is not one for those easily frustrated or short on patience.

3 Normally in implementing GIS technology the readiness of the organization to take on the technolgy is overestimated, while the time and effort required for a successful and sustainable implementation is underestimated.

4. For a successful GIS application, there should be an understanding of the p. es and coof mapping and more important is an understanding of the application for which it is to be used. As an example in case of WRMGIS an in-depth understanding of irrigation management of the area was essential to develop the system.

5 Although GIS technology is now realized in many developing countries as technical necessity, but from political and financial perspective, GIS technology is a nice to have luxury. The financial support for the cost of sustainability, and cost of data quality maintenance, will not move it easily to be a cannot without tool on the national scale.

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WPG Crop and Soil Inventory GIS

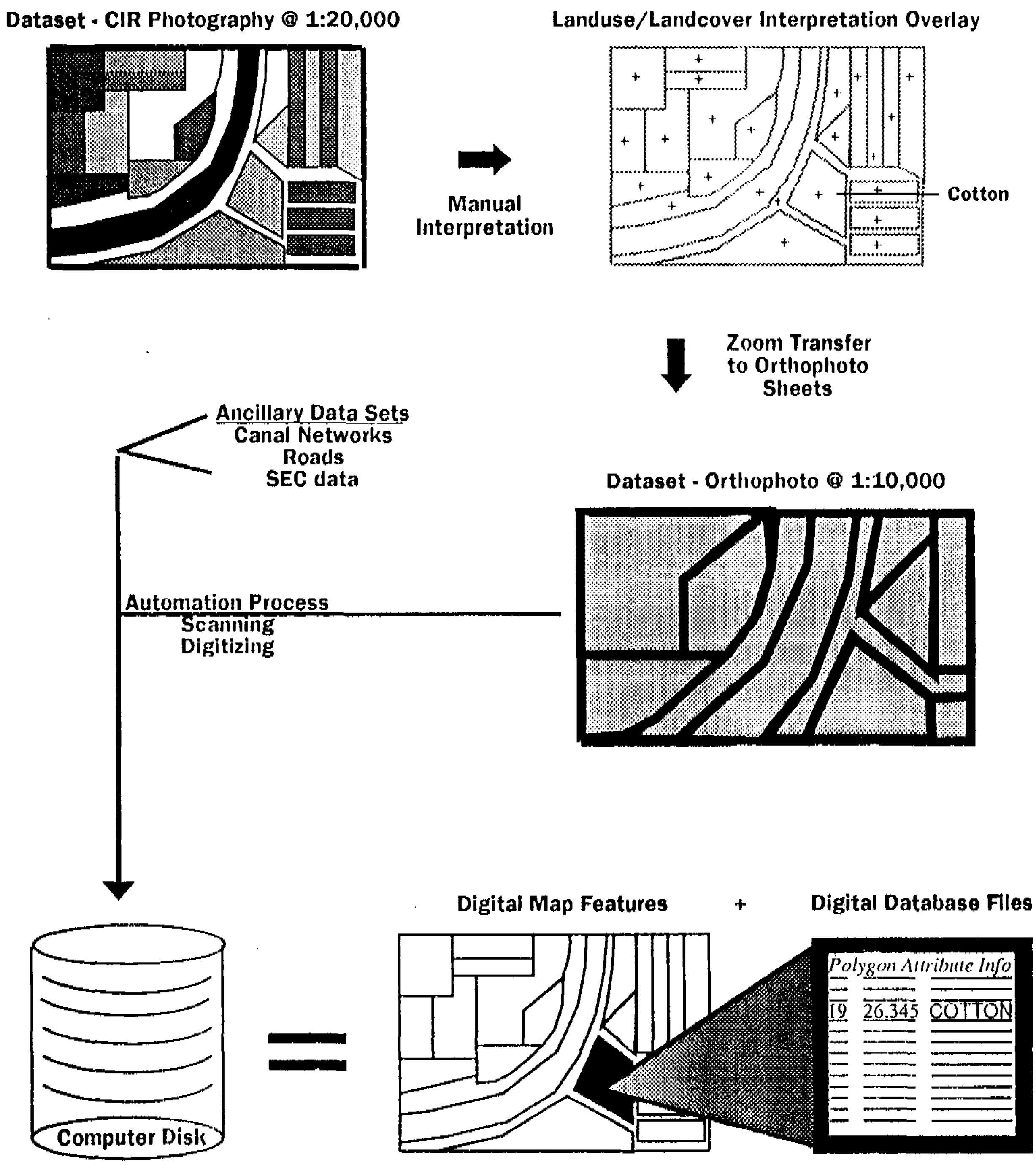

Fig. 1

Concept

GIS — Water Resources Model Integration

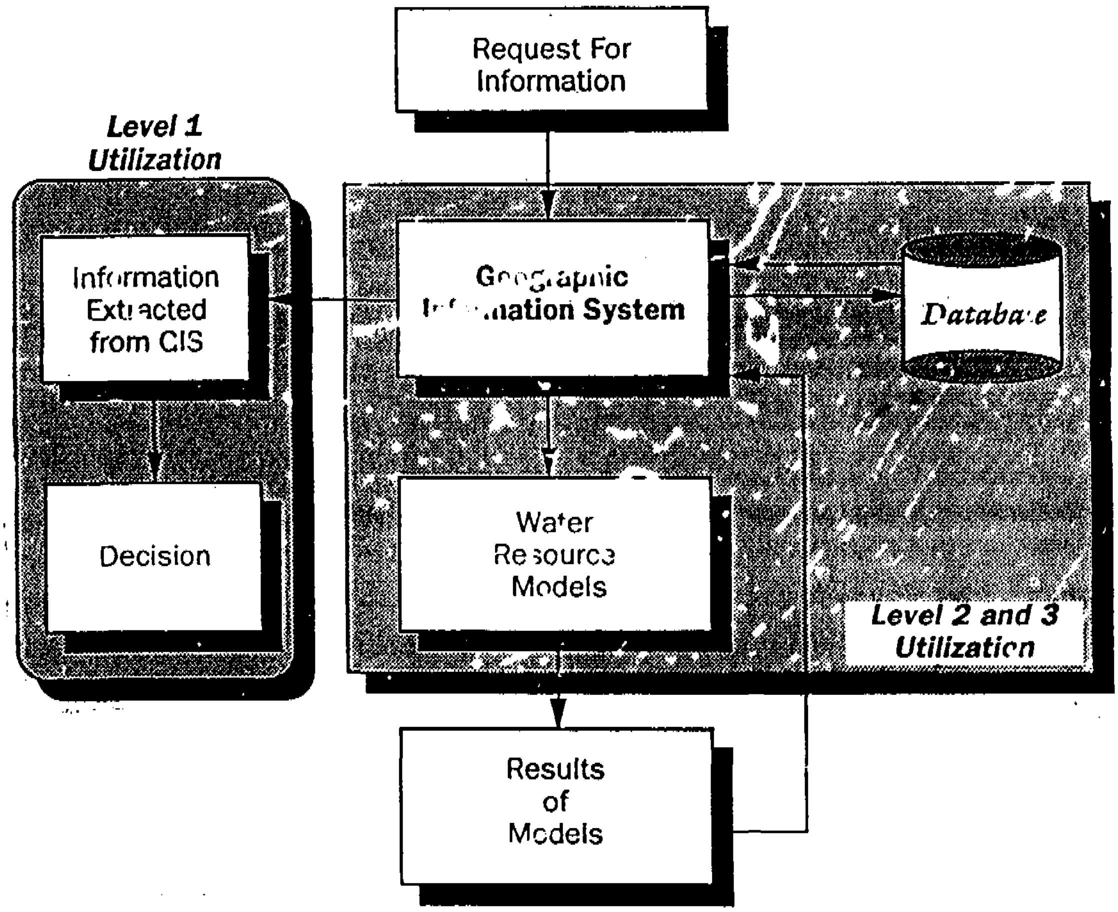

Fig. 2

STRATEGIC WATER PLANNING: AN EXEPCISE CASE STTIDY: EGYPT[1]

Raouf F. Khouzam[2]

Today's Egypt owes its existence to God's gift: water. The presence of this natural resource in this particular place was a prerequisite for the birth of one of the earliest civilizations on earth. Because of it's endurance, the Egyptian civilization outlived it's contemporaries. This fact stays true in the future: Egypt's survival depends in the first place on how this precious endowment is managed today and tomorrow. A glance on the Egyptian media clearly reflects awareness of the issue. A reading of the specialized reports signifies the efforts made by the Ministry of Public Works and Water Resources (MPWWR) is making. Nevertheless, there is still a word to be said.

Sustainability of water resources requires thorough understanding of the present situation. This paper identifies the main variables relevant to water planning and discusses pertaining issues with Egypt's water resources used as a study case. The situation of water resources reviewed with special emphasis on water availability, constraints and needs as well as efforts made by MPWWR to counteract present problems.

Main findings indicated that many important variables are not included. Furthermore, policy tools relied on structural works and forced quotas to run a complex system with rigid supply --even to handle severe drought crisis, rising needs, deteriorated irrigation-drainage network, lack of proper utilization, and degenerating water quality. Such situation requires a cognizant policy that interweave structural measures with effective economic tools.

1. PRESENT AND FUTURE SCENARIOS

The water budget of 1990 (table 1) shows a total available water of 63.5 billion m^3 (BCM). The nature of that magnitude deserves some clarification. It comprises fresh and reused waters.

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[1]This paper draws on section III of a report prepared by the author for the Ford Foundation and Minia University in July 1994 titled "A Reading in Egypt's Water-Land File."

[2]Resource Economist, the Ministry of Land Reclamation. The opinions expressed in this paper are the author's personal views; they do not necessarily represent those of any of the organizations he is affiliated with.

Item	1987	1990	2000	2025 Scenarios		
Scenario No.				A	B	C
Available Water Nile water	55.5	55.5	55.5	57.5	57.5	57.5
Aquifer water	0.5	0.5	2.5	3.5	3.5	3.5
Groundwater	2.3	2.6	4.9	2.6	4.9	3.6
Reused drainage	3.5	4.7	7.0	4.7	8.0	5.0
Recycled sewage		0.2	1.1	1.5	2.5	2.0
Total available	61.8	63.5	71.0	69.8	76.4	71.6
Water Uses Irrigation	50.2	49.7	59.9	43.5	49.7	46.6
Municipal	3.3	3.1	3.1	9.6	14.6	10.8
Industrial	2.5	4.6	6.1			
Navigation	2.0	1.8	0.3	0.3	0.3	0.3
Hydroelectric[1]		1.0	0.0	0.0	0.0	0.0
Evaporation[2]	2.0	2.0	2.0	2.1	2.1	2.1
Total uses	60.0	62.2	71.4	55.5	66.7	59.8
Surplus	1.8	1.3	-0.4	14.3	9.7	11.8

Sources:
1987: MPWWR/Ford Foundation 1988:12-13.
1990 & 2025: Abu Zeid and Rady 1991:50-51(tables 9&10),p.62 (table 12).
[1]Added from Abu Zeid 1990.
[2]Added from Abu Zeid and Rady 1991:52(figure 10).

Table 1: Water budgets for the years 1987, 1990, 2000 and the committee scenarios for 2025.

Total **fresh** water is 56 BCM of which 55.5 BCM is Egypt's share of the Nile's water and 0.5 BCM is an estimate of water extraction from aquifers. The remainder (7.5 BCM) is nothing but reused quantities of the **fresh** volume. Actually, the increase in reused water is the main source of the rise in total available water. The water budget of 1987 was 61.8 BCM increased by 1.7 BCM to reach 63.5 BCM in 1990. The increase in total available water came mainly from the rise in reusing drainage and sewage water. Similarly, the increase in available water in future scenarios relies on raising water recycling.

Table 1 also indicates that about four fifth of total available water is used for irrigation. Comparatively, other uses are of minor magnitude. Balancing available water with the needs, a small surplus is obtained although water shortage is used to be observed in different old- as well as new-land regions (DDC 1985, 1986, and 1987; Skold et al. 1984, and Bowen and Young 1982). One explanation of this discrepancy is that observed shortage is a result of unequal water distribution; if maldistribution is corrected such shortage will be eliminated with the same quantity of water. Another interpretation is that the water budget is balanced using unverified secondary sources (reused drainage water and extraction from an unknown number of shallow wells). Such figures could be over or below the right magnitudes; for example in 1987 reused drainage water is estimated at 3.5 BCM dropped to 2.8 BCM in 1989 while the trend is increasing. A third explication is the lack of accurate measurement on both sides of the water budget. For example Biswas noticed that the estimated irrigation requirement is nothing but the difference between releases at Aswan and other outflows and usages. Hence, estimated irrigation requirement includes, in addition to crop consumptive use, any unaccounted-for water (Biswas 1991:42). The above factors altogether suggest that it is equally possible that a shortage, instead of a surplus, could have taken place. Anyhow, regardless whether it is a surplus or a shortage, the issue of water planning certainly requires immediate attention.

1.1. Water Policy

The guidelines of a water policy is drawn by a "Supreme Ministerial Committee" (Abu Zeid and Rady 1991:IV). Accordingly, the Ministry of Public Works and Water Resources (MPWWR) allocates available water among the economic sectors via a quantity-rationing approach. The last few decades have witnessed a sharp change in the main features of water policy. For centuries, flood was used to require the mobilization of resources; the main concerns of policy makers were flood protection. Other concerns were the expansion of the irrigation-drainage network to serve greater areas, enlarge the storage capacity as a precaution against drought and as a devise to balance the distribution between the flood and the dry seasons, intensifying agriculture, and,in addition, power generation.

Naturally, structural works were the most suitable measures to achieve the sought objectives.[4] Subsequently, a series of dams and barrages were built and the irrigation-drainage network has been expanded tremendously from 9.8 thousand km in 1885-89 (Radwan 1974:36) to reach, in 1990, 31 thousand km of public canals, 80 thousand km of field channels, and 17 thousand km of public drains (ISPAN 1990:2). Such complex system cost Egypt a total of LE 730 million at 1960 prices accumulated during the period 1877 to 1967 (calculated from Radwan 1974:34). Investments in new structures and system rehabilitation have been continuing.

More recently, a number of factors forced a significant shift in water policy. On the supply side, a decade of drought brought down the average river yield down to an average of 48.6 BCM (Abu Zeid and Rady 1991:36,47 and Abu Zeid 1992-b:12-14). The drought was accompanied by a complete stop in the works on the Jonglei-I project which would have provided Egypt with precious 2 BCM. On the demand side, an ambitious program to reclaim 150 thousand feddans annually has widened the gap between supply and demand *a fortiori*. These factors necessitated a tight water policy.

Measures of the tight water policy comprised (Abu Zeid and Rady 1991:47-48 and Abu Zeid 1990): (1) the elimination of special water release for the purpose of generating hydroelectric

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[4]Structural works include the Delta barrage, Aswan dam, Assiutt barrage, Zifta barrage, Esna barrage, Sennar dam, Nag-Hammadi barrage, Edfina barrage, Gabal El-Awlia dam, the High Aswan Dam (Radwan 1974:22-26 & 261).

power, (2) the construction of Isna barrage to save water release made in order to reduce the head increase resulting from river bed erosion, (3) prolongation of the winter closure period, (4) minimization of water release through Rosetta branch, and (5) modifying the current cropping pattern in order to reduce its water requirement.

Obviously, top-dictated structural measures are still the main policy vehicles. This is a normal extension of the school of thought prevailed before building the High Aswan Dam (HAD). That school has not lost its momentum. As a matter of fact, the very nature of water resources in hydrological societies feeds that momentum. Nonetheless, non-structural measures are needed more nowadays to induce water conservation and optimization are not effectively utilized.[5]

Notwithstanding, quantity-rationing is still the allocation vehicle. Although cheap in terms of its explicit cost, suffers a number of serious shortcomings. First, it fails to provide decision makers at various hierarchical levels with signals necessary to induce optimization and/or conservation at the system's level whether in time or space dimensions. Second, if every user is forced to take whatever quantity available to him, excess demand (or supply) gap will not be revealed. As discussed above, water shortage has been reported in many regions though current water budgets showed a surplus. As such, system users are motivated to race to appropriate whatever quantities they can get in order to secure their needs. Hence, rationing fails to induce water users to optimize or conserve on water appropriation. Third, quantity-rationing approach fails to internalize externalities (especially pollution); this undermines the sustainability of the water resources system and other dependent activities. What makes these reservations more critical is the forecasts for the year 2025.

1.2. Formal Water Scenarios

An early systematic effort to forecast future water budgets was that of the Water Master Plan (WMP). Since its inception in 1978, it comprehensively addressed various components of water resources in about 60 volumes. A continuation of WMP is the work of an interdisciplinary committee formed by MPWWR. The committee presented three scenarios for the year 2025 (shown in table 1). The three scenarios are based on a number of assumptions which are similar with respect to some elements but different in others. All scenarios anticipated the completion of Jonglei I by the year 2025. As a result, Egypt's share of the Nile water is assumed to rise by 2 BCM. Furthermore, the extraction of aquifer water is assumed to increase seven folds its 1990 level. As for the other supply elements, the scenarios assumptions differed significantly. The quantity of extracted underground water is held constant in scenario ***A***, and allowed to increase by 88% and 38% in scenarios ***B*** and ***C***; respectively. Similarly, the quantity of reused drainage water is kept fixed in scenario ***A***; those of scenarios ***B*** and ***C*** are left to rise by 70% and 6%; in

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[5] In this text, optimization is used to indicate using the same quantity of water to produce more in quantity or in value; conservation means producing the same level of output with less water.

order.[6] Recycled sewage water is assumed to rise by 6 to 12 times. Given this set of assumptions, the scenarios forecast that total available water will range from 70-76 BCM.

The three scenarios are biased towards optimistic hypothetical situations leaving aside realistic pessimistic possibilities. As pointed out in table 2, the rise in total available water has unverifiably been assumed to exceed that of total water use. When total water uses was assumed to drop to its minimum in scenario ***A***, the corresponding total available water had increased slightly. On the other extreme, when total water uses was assumed to rise to its maximum in scenario ***B***, total available water was assumed to reach its greatest value. Thus, the way the scenarios are set up implies that both total available water and its counter part the total water use are under the full control of MPWWR such that if the first drops the latter can be reduced and *vise versa*. Such critical tacit assumption drastically narrows the planning horizon of water resources and deprives the decision makers from perceiving and planning for other realistic possibilities.[7]

	1990	*Scenarios*		
		A	*B*	*C*
Total Available Water	*63.5*	*69.8*	*76.4*	*71.6*
Change from 1990		*6.3*	*12.9*	*8.1*
Change %		*10*	*20*	*13*
Total Water Use	*62.2*	*55.5*	*66.7*	*59.8*
Change from 1990		*-6.7*	*4.5*	*-2.4*
Change %		*11*	*7*	*-4*
Surplus	*1.3*	*14.3*	*9.7*	*11.8*
Change from 1990		*13*	*8.4*	*10.5*
Change %		*1000*	*646*	*808*

Source: Calculated from table 1.

Table 2: Biasedness of committee scenarios.

In addition to the above shortcomings, the committee's scenarios did not address a number of important issues some of which invite a rather optimistic attitude, others call for pessimistic concerns. On the optimistic side there is: (a) the possibility of executing upper Nile projects other than Jonglei I, and (b) water savings that could be made as a result of the inevitable encroachment of the old-land. On the pessimistic side, the possibility of a decreasing trend of annual Nile yield as a result of a south shift in the rain zone has been ignored. Taking these aspects into consideration would allow a more flexible planning practices; this issue is the focus of the following section.

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[6]Surprisingly, groundwater and reused drainage water are allowed to drop in the year 2025 below the level anticipated in the year 2000. This implies a contradicting trend!.

[7]Assumptions underlying to various water uses are discussed below in the corresponding sections.

1.3. Alternative Scenarios

Future scenarios are nothing but a graduated shades of optimism and pessimism blends. A scenario with rather optimistic shade anticipates success in efforts to improve the system as well as positive behavioral attitudes of the users and bureaucrats. Clearly, a pessimistic version assumes the opposite. In the light of that definition two scenarios are derived and presented in table 3. Discussion concentrates on Nile and other surface resources.

Item	*Optimistic* <u>Y</u>	*Pessimistic* <u>Z</u>
<u>*Available Water*</u> *Nile water*	55.5	50.3
Upper Nile projects	9.0	0.0
Aquifer water	3.5	0.5
Groundwater	4.9	2.6
Reused drainage	8.0	4.7
Industrial effluent	9.5	18.9
Recycled sewage	1.0	2.0
Total	9'.40	79.00
<u>*Water uses:*</u> *Irrigation:old land*	35.1	41.3
Irrigation:new land	17.0	17.0
Municipal	6.0	12.0
Industrial	9.85	19.7
Navigation	0.0	1.8
Hydroelectric	0.0	1.0
Evaporation	0.45	0.02
Discharge to sea	10.0	10.0
Total	78.40	102.82
Surplus	13.00	-23.82

Table 3: The author's optimistic and pessimistic scenarios.

1.3.1. Supply Sources

Chances to increase available water is very limited indeed. They comprise raising Egypt's share in the Nile's water via upper Nile projects, assessment of safe yield of extraction from aquifers and underground water, raising the rate of water recycling, improving the efficiency of various water uses, minimization of releases especially made for non-consumptive uses, and more sensitive response to land encroachment. The following sections address each of these elements.

1.3.1.1. <u>The Nile: international aspects.</u> The great Nile river basin has an area of about 3 million Km^2 which impinges upon 10 African countries: Burundi, Rwanda, Tanzania, Uganda, Zaire, Central African Republic, Kenya, Ethiopia, Sudan, and Egypt. Of these regions, the Ethiopian highlands is the source of nearly 85% of the water of both Egypt and the Sudan. Historically, Egypt's water arranged by a number of agreements: a protocol signed in 1891 between the United Kingdom and Italy (on behalf of Egypt and Ethiopia; respectively), in 1902 between Britain and Emperor Menelek II when Italy left Ethiopia, and in 1925 with Italy when Ethiopia fell under its domination again (Waterburry 1979:74-75

Water is distributed between Egypt and the Sudan according to 1959 "Agreement for the Full Utilization of the Nile Waters." Assuming a mean annual discharge of the Nile at Aswan of

84 BCM, 10 BCM is assumed lost in High Dam Lake, Egypt receives 55.5 BCM, and the Sudan 18.5 BCM (Waterburry 1979:72-73; and Abu Zeid and Rady 1991:5).[8]

1.3.1.2. The Nile: upper Nile projects. Accordingly, it is assumed that the share will stay constant until the year 2025. However, it is allowed to rise on the ground that all upper Nile projects are carried out as discussed in the next section; this is a component of the optimistic scenario ***Y*** (table 3). On the other hand, a drop below the agreed upon share is assumed under a set of pessimistic conditions discussed and shown in scenario ***Z***.

With all Nile basin countries growing, their needs for water is increasing. Hence, no country will be able to give concessions to another. As such, the only way to raise Egypt's share of the Nile water, in addition to that of other countries, is through the implementation of upper Nile projects. Action on such projects is subject to political negotiation in the first place, and the political stability in the project's region.

Complications involved in the initiation and the completion of upper Nile projects were behind the failure of some of the WMP projections. Phase I of the Jonglei project was anticipated to be completed by 1985, Mashar Marches by 1990, Jonglei phase II and Bahr El-Ghazal projects by the year 2000 (WMP 1980:74-76,101). Uncertain as they are, MPWWR committee dismissed in all its scenarios the completion of any projects other than the first phase of the Jonglei canal.

> *Jonglei Canal I: Started in 1978 and was abandoned in 1983 due to security problems. It was supposed to yield 4 BCM by 1985.*
>
> *Jonglei Canal II: 3 BCM*
>
> *Mashar Marches: Total loss in the Mashar swamps is 10 BCM. Conservation schemes saves 4.4 BCM at the White Nile or 4 BCM at Aswan*
>
> *Bahr El-Ghazal: A sub-basin of the Equatorial Plateau. Its discharge in a normal year is 14 BCM of which only 6 BCM reaches the White Nile at Lake No. The conservation project saves 9 BCM at Malakal or, equivalently, 7 BCM at Aswan.*
>
> *Thus, the total at Aswan is 18 BCM to be equally shared with the Sudan.*
>
> *Source: Abu Zeid 1992-b:4*

Text Box 1:Upper Nile Projects.

Though the committee's tendency is supported by real life experience and by the on-going conflicts, it is not unrealistic to be optimistic about the completion of all upper Nile projects by the year 2025. This attitude is based on a number of factors: a) available time span is long enough to negotiate and implement those projects, b) improvement in the relationship between Egypt and most of the Nile basin countries, c) the continuing successful cooperation among Nile basin countries since 1967 to implement a comprehensive hydrometeorological survey for the Nile basin (Ezzat and Ouf 1993), and d) the surviving UNDUGU which Inga-Aswan electrical connection is an outcome (Henery and Androsov 1989:3). Cooperation in the area of hydroelectric power would eventually lead to cooperation in the area of water management. Nevertheless, as not all the members are Nile basin countries, special arrangements have to be negotiated outside UNDUGU

[8]Earlier, the 1929 agreement provided the Sudan and Egypt with 4.5 and 47.5 BCM; respectively.

(ECAUN 1992). Assuming Upper Nile projects water gets equally split between Egypt and the Sudan, brings Egypt's water share up by 9 BCM as shown in scenario ***Y***, if implemented.

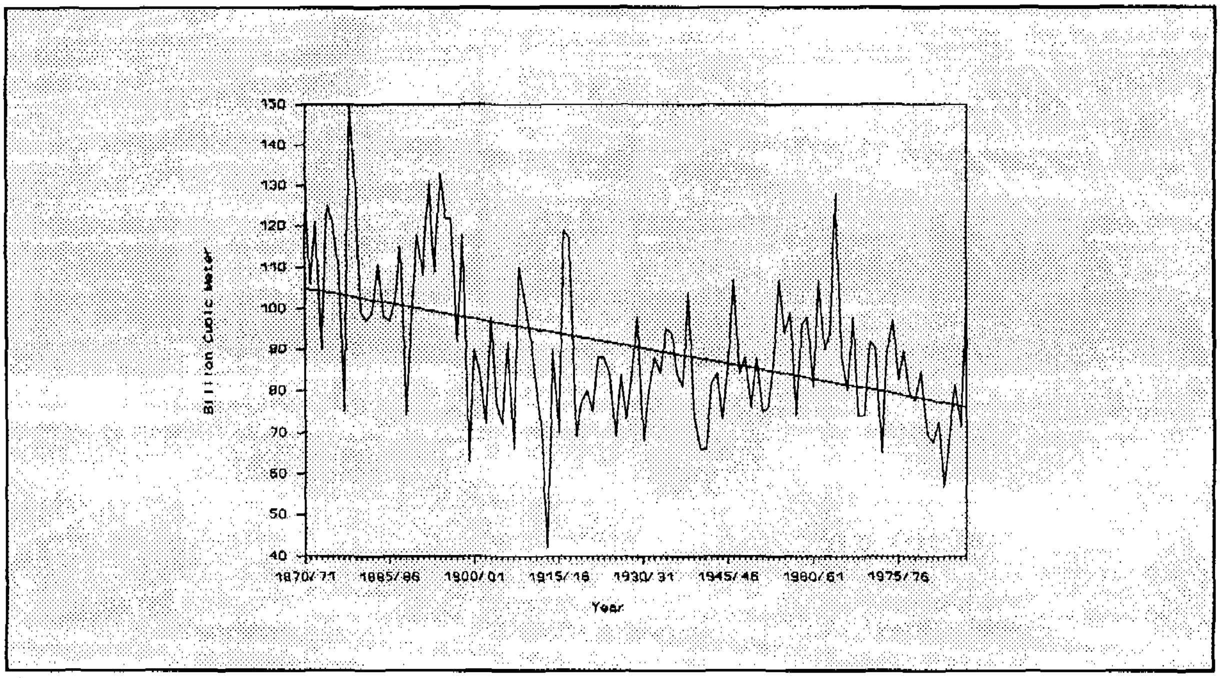

Figure 1: Decreasing trend of the Nile's annual natural yield.

1.3.1.3. <u>The Nile: falling yield</u>. Taking an optimistic extreme can only be balanced by exploring a pessimistic one as well. It is known that the annual river yield fluctuates significantly from a year to another depending on the rainfall in the upper Nile basin. Data of the annual river yield over 116 years suggests a decreasing trend over time.[9] Another study of the same series took the percentage of long-term mean as a reference point. It showed that the river flow was high during the period 1871-1905, less than the long-term mean during 1905-1965, and significantly lower during the post 1970 period (Abu-Zeid and Rady 1991). A third proposition suggested that river flows during the post 1970 period is a reoccurrence of dry conditions prevailed in the 19th century (Biswas 1991:70,72). These ideas are in agreement with a proposition that a climatological change is shifting rainfall in the upper Nile region to the south (Hulme 1990:59). If that trend is accepted, the Nile's water may drop to 50.25 BCM by the year 2025. This idea is adopted in scenario **Z**

1.3.1.4. <u>Improving irrigation efficiency</u>. The average cost of obtaining an increment of one BCM via upper Nile projects is estimated at LE 300-350 million (Abu Zeid 1992:8). Such high cost

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[9]The time trend presented in figure 1 is of the form $Y=a+bX_t$; it is not known whether the data has been adjusted for silt deposit or erosion. Some estimates of the slow accretion, a regular feature of the Nile region, amount to 12-13 cm/100 years (Evans 1990:13).

makes other ways such as improving irrigation efficiency more attractive. The current irrigation efficiency is in the vicinity of 55% with some field studies estimating it as low as 43% (MPWWR 1990-c:3.3).

Factors underlying the system's low efficiency are summarized in a number of studies (Abu Zeid 1992:4; Abu Zeid and Rady 1991:36; MPWWR 1991-a:17-19; MPWWR 1991-b:3.2; MPWWR 1990-c:3.2; MPWWR 1990-d:3.3-3.4; MPWWR 1990-e:16-29; MPWWR 1990-f:20-26; MPWWR 1990-g:i-ii; MPWWR 1988:III.4-III.5). Those factors are either the result of the system's deficiencies, or due to farmers behavior. With respect to the first category, the system suffers insufficient structures, poor repair and maintenance, flat canal grades, loose observation of rotation schedules, and inadequate monitoring of water levels, withdrawals, discharges or quality.

Farmers themselves have their role in depressing the system's efficiency. Although the system is designed for twenty-four-hours irrigation, farmers tend to avoid night irrigation. As a consequence, considerable quantities of water overflow at canal ends. Besides, poor land levelling raises withdrawals to cover high points in the field. To make it even worse, inequitable water distribution induced farmers to over-irrigate as a precaution against uncertain water delivery. Moreover, negligence of clearing water courses leads to significant losses. Those losses are magnified by the misuse of canal banks which result in widening canals cross section. Furthermore, large variations in crops coupled with land fragmentation make it very difficult to follow a thrifty irrigation schedule.

Recognizing the sensitivity of the situation, MPWWR has been carrying out a large-scale ambitious scheme to rehabilitate the irrigation-drainage system, and to improve its management. To handle farmer-related aspects, MPWWR initiated "Egypt Water Use and Management Project" (EWUP) which findings were materialized in the Irrigation Improvement Program (IIP).[10] IIP was originally planned to renovate local system's components, develop Irrigation Advisory Service to help farmers to adopt modern water management techniques, and organize Water User Associations. If successfully implemented, IIP is expected to raise land productivity by 30% and, meanwhile, makes water savings of 10-15%. If really achieved, then IIP will satisfy the charming formula: produce more using less resources. Unfortunately, IIP is progressing at a rate below the planned one (Abu Zeid 1992:4). Target area is found unrealistic given the time and budget constraints (ISPAN 1990).

IIP is just one component of a greater comprehensive project known as the Irrigation Management Systems Project (IMS) which was designed to deal with both system- as well as farmer-related obstacles. It consists, in addition to IIP, of a number of components: Structural

[10]EWUP was started in 1976 in three different regions and was completed in 1985. Its purpose was to develop and demonstrate replicable farm water management practices which raises system's efficiency and, at the same time, boosts agriculture growth (USAID 1987:46).

Replacement project, Preventive Maintenance, and Main Systems Management, Planning Studies and Models, Professional Development, Project Preparation, Survey and Mapping, in addition to other main components (ISPAN 1990:1).

IMS components which focus on improving the system itself are the Structural Replacement, and the Preventive Maintenance/Channel projects. They are directed to the rehabilitation of small and medium structures, and the reduction of maintenance cost through improved management. The Main Systems Management component aims at building an information system (e.g. telemetry, data management) to support critical decision making. To utilize the collected information, the Planning Studies and Models component has been designed to support modeling efforts with the assistance of the U.S. Bureau of Reclamation to forecast the inflows to Lake Nasser, to estimate irrigation demands and return flows down to canal level (ISPAN 1990:15-19).

Given the above efforts, scenario ***A*** assumed that irrigation efficiency will improve to 75% by the year 2025. Hence, irrigation share drops from the 1990 level of 49.7 BCM to 43.5 BCM making a saving of 6 BCM. Scenario ***B*** takes the other extreme assuming no change in field irrigation efficiency on the ground that socioeconomic constraints will impede any improvements. As such, irrigation requirement will stay at its current level. Scenario ***C*** takes a middle ground; irrigation efficiency will reach 65% bringing down irrigation requirement to 46.6 BCM and making a saving of 3 BCM. It is worth mentioning that water savings in scenarios ***A*** and ***C*** are based of the full implementation of IMS.

1.3.1.5. Reuse of drainage water. Another source for increasing available water is more reuse of drainage water. While about 12 BCM was discharged in the sea (Abu Zeid 1992-b:6), only 2.8 BCM was reused in 1989 (Abu Zeid 1989:33). This quantity rose to 4.7 BCM in 1990 and is expected to reach 7 BCM by the year 2000, then to 8 BCM by the year 2025 according to scenario ***B***. However, the other two scenarios almost assumes no increase in reused drainage water above 1990 level!.

Year	*Flow at HAD*	*Drainage to Sea*		
	BCM	*BCM*	*Salinity mmhos/cm*	*% to Flow at HAD*
1984-85	*56.40*	*14.30*	*3.71*	*25.35*
1985-86	*55.52*	*14.07*	*3.72*	*25.34*
1986-87	*55.19*	*13.59*	*3.59*	*24.62*
1987-88	*52.86*	*12.27*	*4.12*	*23.21*
1988-89	*53.24*	*12.03*	*4.26*	*22.60*

Source: Abu Zeid 1992-b:8

Table 4: River flow at HAD and water discharged to the sea.

According to 1990 budget, the total drainage water is 18.2 BCM;[11] it represents about one third the Nile's water. The total amount of drainage water depends on: the magnitude released at

[11] 13.5 BCM is discharged into the Mediterranean Sea and Northern lakes plus 4.7 BCM reused drainage water.

HAD, the prevailing cropping pattern, and the system's efficiency (Abu Zeid 1992-b). The amount of reused drainage water is governed by:

* the quality of drainage water. As shown in table 4; salinity of drainage water is indirectly related to its quantity. Hence, system's improvement is expected to reduce the quantity as well as the quality of drainage water.
* the availability of fresh water to dilute the amount of drainage water to be reused,
* the availability of funds to build the pumping stations required to lift and mix drainage water plus other operating costs. That brings up another point, the economic feasibility of investing in reusing drainage water,
* loaded with chemicals and pesticides residues the long-term impact on the environment, and
* the amount of drainage water which must be released to the sea to leach the salts coming in the Delta region (Abu Zeid 1992:6-9).

None of the formal scenarios provided the quantity of water need to be discharged in the see for technical consideration (leaching the Delta and prevention of salt intrusion). Drainage requirement has been included in scenarios ***Y*** and ***Z*** (table 3) on the ground that it should not be less than 10 BCM.[12]

1.3.1.6. <u>Groundwater</u>. This term refers to the Valley and the Delta shallow reservoir which is recharged by seepage from the irrigation-drainage system. A study by the Research Institute for Groundwater indicates the presence of a high capacity reservoir of 500 BCM with average salinity of 800 ppm underlie the Nile valley and delta. The current annual rate of extraction of groundwater for all uses is estimated at 2.6 BCM. This can be increased to 4.9 BCM which is believed to be equal to the rate of recharge (Biswas 1991:38).

An interesting feature of the extraction of groundwater is that it generates a positive externality in the form of reduction in the level of water table. Hence, solving water logging problems (MFF 1988:14 and Biswas 1991:38). Nevertheless, the conjunctive use question may represent a negative externality: withdrawal may raise seepage further reducing that way available surface water. It seems that this issue has not received enough attention.

Since the available information does not provide a ground to make variations from the committee's scenarios, the highest and lowest values of the committee's scenarios are adopted in scenarios ***Y*** and ***Z***; in order.

1.3.1.7. <u>Recycling sewage water</u>. The reuse of treated Cairo sewage water was initiated in 1915 in the eastern desert to cultivate 2,500 feddans (Biswas 1991:46). Recycled sewage water may

[12]Total drainage water is estimated at 18 BCM, maximum recycled water given in the committee scenarios is 8 BCM. It is, hence, assumed that it is a must to discharge the rest into the sea.

reach 2.5 BCM pending upon the completion of sewage treatment plants in Cairo (its capacity is expected to reach 1.93 BCM by 2010), Alexandria (2.2 BCM), and the rest of the country which will produce up to 4.9 BCM (Abu Zeid 1992:4). This potential source is not considered in the committee's scenarios!. It has been added to scenario ***Y***. In scenario ***Z***, the 1990 level is kept unchanged.

A couple of advantages is associated with the use of properly treated wastewater. Nutritionally, it is generally more useful for irrigation than fresh water. Dried sludge can effectively be used as a soil conditioner for agricultural land; during 1988-89 about 46,000 m^3 of dried sludge was sold at LE 138 thousand (Biswas 1991:47-49).

1.3.2. Water Needs

Within Egypt, water is distributed and drained through a complex network. As a consequence, there is a time lag of 10 days between water release from the High Aswan Dam and the main drainage outlets. The time lag limits the system's responsiveness to unexpected changes that require immediate change in water release at points of control. Thus, investing in enhancing the capabilities of projection of water needs is worthwhile.

Water needs include irrigation of old- and new-land, municipal and industrial uses, non-consumptive uses for navigation and for power generation.

1.3.2.1. Irrigation: old land.[13] The estimation of per-feddan water requirement is a complex issue. It differs according to the cropping pattern, soil type, weather, irrigation technique ...etc. Furthermore, it requires accurate information about the irrigated area, cropping pattern, conveyance losses. A way around these problems is to adjust committee's estimates.

Water saving resulting from old-land encroachment has not been considered by the committee. Although encroachment is prohibited by law, it is still practiced legally and illegally. Legally, official institutions are obligated to use agriculture land for the development of rural Egypt. Illegally, the loopholes are skillfully being utilized by violators, violations are overlooked in order to avoid tension, favourism and nepotism play a principal role, a lengthy bureaucratic procedure to take effective action against violators, the value of the fine is small when compared to the significant gains a violator makes from allocating agriculture land to urban uses (AlMussawar 1993). A conservative estimate of the loss of fertile soil to urbanization and rural development projects is about 30 thousand feddans per year (Biswas 1991:18-19).

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[13] Old land in the committee's document includes new land being cultivated until 1990.

Requirement m^3/f/yr	Region	Source
6900	old land	calculated from Abu Zeid & Rady 1991:62
5500-6050	new land	calculated from Abu Zeid & Rady 1991:59-60
5065	new land	WMP 1980:8
6051	new land	Abu Zeid & Rady 1991:59
6590-11200	new land, field crops	LMP 1986:26[1]
5450-8040	new land, citrus	LMP 1986:26
4150-6230	new land, grapes	LMP 1986:26
4150-6230	new land, olives	LMP 1986:26
9300	upper Egypt	Goueli & El-Miniawy 1993:35
7300	middle Egypt	Goueli & El-Miniawy 1993:35
6720	lower Egypt	Goueli & El-Miniawy 1993:35

[1]Lower and upper ranges of LMP estimates correspond to upplands in Lower & Upper Egypt; as for lower lands leaching requirement has to be added.

Table 5: Some evapotranspiration estimates.

Given the area encroached annually, water saving depends on the water requirement that area. Some old-land per-feddan water requirement is shown in table 6; it ranges from 6720 to 9300 m^3. Adopting an average of 8000 m^3/feddan, encroachment saves 240 million m^3/year. This saving is cumulative; i.e. total annual saving reaches 8.4 BCM by the year 2025. Hence, old-land irrigation requirement would take a downward trend even if efficiency does not change. Accordingly, the old-land irrigation requirement will drop to 35.1 BCM if irrigation efficiency improves to 75% (scenario **Y**) or will fall to 41.3 BCM if it stays at 55% (scenario **Z**). Nevertheless, such savings cannot be realized unless the planned control improvements are achieved such as the Nile telemetry to practice effective control over water flow in various canals.

1.3.2.2. <u>Irrigation: new land</u>. The Land Master Plan (LMP) estimated total reclaimable land at 3428 thousand feddans (LMP 1986:vi). Of this area, about one million feddans has been reclaimed by 1992 (MOA&LR 1993:28). The remainder is, thus, 2.5 million feddans still to be reclaimed. Table 6 shows a number of a number of per-feddan water requirement. The wide variation is clear; it ranges from as low as 4200 m^3 to as high as 11200 m^3. Taking 7000 m^3/feddan as an average, irrigation requirement for this area is about 17 BCM; this same amount is used in both the optimistic and pessimistic scenarios. The committee scenarios did not consider new lands; land areas to be reclaimed according to the available water surplus.

1.3.2.3. <u>Municipal use</u>. Increase in municipal use emanates from population increase, income increase, hygienic awareness, and rural development. Nevertheless, available data dictated the use of physical magnitudes.

A comparison between 1987 and 1990 uses indicates a drop of 0.2 BCM over the three years in spite of the fact that the population increased, level of income rose and potable water was extended to many rural areas. However, during the period 1990-2000, it is assumed that increase in use will be met by the water saved through improving efficiency use from 50% in 1990 to 90% in 2000. Taking the 1990 level of municipal use of 3.1 BCM; 50% loss means another 3.1 BCM is lost back to the system.[14] Reducing waste to 20% by the year 2000 and, at the same time, holding gross municipal use constant as shown in table 1, indicates that consumptive use is raised to 4.96 BCM or, equivalently, a rate of increase of 0.186 BCM/year. If this rate continues, then by the year 2025 municipal consumptive use alone would be 7.75 BCM.

Another approach to estimate future use is to use WMP projection. Annual municipal use is estimated at 1.5 BCM in 1976 projected (at the time of report preparation) to reach 2.08-2.1 BCM in 1982, and 3.3-5.4 BCM in 2000 (calculated from WMP 1981-a:6.2-6.4). Given that information, average annual rate of increase from 1976-1982 was 97-103 million m^3, from 1982-2000 it was 68-182 million m^3. Comparing the lower rates in the two periods, it is taking a downward trend. Contrarily, the upper limits are taking an upward trend. The lower limit of 1982-2000 is adopted to calculate the optimistic use and the higher rate is followed to estimate the pessimistic municipal use. Accordingly, consumptive municipal use is expected to reach 5 and 10 BCM. Adding 20% waste brings these to magnitudes up to 6 BCM in scenario **Y**, and 12 BCM in scenario **Z** with 1 and 2 BCM are returning to the system.

1.3.2.4. Industrial use. The rise in industrial use is attributed to the growth of the industrial sector, kinds of industries and types of water use (cooling, washing,...). Based on extrapolation of the 1980 WMP survey; the committee estimated the industrial use in 1990 at 4.6 BCM (Abu Zeid 1992:6) and expected to reach 6.1 BCM in 2000 (average annual rate of increase 0.15 BCM). According to that rate of increase, industrial use is anticipated to reach 9.85 BCM.

WMP estimated industrial use in 2000 at 9.7 BCM of which the consumptive use is only 4% (WMP 1981-b:2.1); average annual rate of increase is 0.4 BCM. Given that rate of increase, by 2025 industrial use is expected to reach 19.7 BCM of which 18.9 BCM will return to the system. The lower projection of 9.85 BCM is considered the optimistic scenario **Y**; the higher projection of 19.7 BCM is assumed in the pessimistic scenario **Z**. In both cases, it is assumed, after WMP, that 96% of the withdrawal will return to the system.

A crucial question concerning municipal and industrial uses is not the quantities withdrawn, because most of it returns to the system, but the quality of returning water and the cost of its treatment for reuse.

1.3.2.5. Non-consumptive uses: navigation and hydroelectric power. By the year 2000, water release especially made for non-consumptive uses will be minimized. Navigational water requirement is expected to drop to 0.3 BCM by better control of the Nile water level as a consequence of the construction the Esna Barrage (Biswas 1991:45). Special release for the

[14] Another efficiency estimate is 60% which is expected to rise to 70% (calculated from Negm 1992:49).

generation of hydroelectric power will be completely eliminated. Complete elimination of these uses are assumed in scenario **Y** but assumed to retain the same level of 1990 in scenario **Z** .

2. SUMMARY AND POLICY RECOMMENDATIONS

Formal water budgets (shown in table 1) are marginally balanced until the year 2000. Scenarios for the year 2025 provide a surplus ranging from 10-14 BCM which allows reclaiming new lands. As demonstrated in table 2, those scenarios tacitly imply that water availability and use are under the full control of MPWWR. Furthermore, the scenarios missed considering a number of key elements: the possibility of executing upper Nile projects (briefly listed in text box 1), water savings resulting from encroachment of agriculture land, and a possible drop in annual river yield (presented in figure 1) as a consequence of regional climate changes. Taking these factors into account resulted in two new scenarios presented in table 3.

The alternative scenarios represent two extremes: an optimistic one which expects a water surplus of 13 BCM by 2025 and, on the other extreme, a pessimistic scenario that anticipates a water shortage of 24 BCM. Water shortage in itself is not the key question. Normally, societies possess endowments less than their needs. The alarming issue is the crafting of suitable institutions to induce appropriate resource management. For centuries, management of water resources have been via structural measures and forced quantity rationing. It is indicated that those means are still the main tools of management. However, the system's low efficiency proves the insufficiency of those tools to generate conservative behavioral patterns. Non-structural measures must be called in before it is too late especially that it takes long time to change behavioral attitudes.

Moreover, future planning exercises should exceed the current phase of physical planning to economic planning that incorporates costs of various actions and their benefits. Timely distribution of water, collecting drainage, and enforcing rules and regulations is an industry which has the same set of conventional inputs as any other industry. It has capital investment, variable cost, OM&R, and labor. Whether the society as a whole pays the bill or users do, costs as well as returns to various scenarios have to be taken into account when ranking is needed.

Last but not least, this paper is an intermediate phase in a larger analytical framework for planning of water resources. This phase addressed physical variables of water planning at the national level. Further work will expand this phase hierarchically into two directions a lower level and an upper one. The lower level disaggregates the national level to regional levels. The upper level encompasses, in addition to Egypt's budget, other nations' budgets to form one Great Nile basin water budget to identify possible future bottlenecks and collective means to resolve them.

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A MODEL FOR EFFECTIVE IRRIGATION DEMAND REDUCTION AND WATER SHORTAGE MANAGEMENT

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ABSTRACT

A special computerized model, the Irrigation Management Information System (IMIS), was developed and implemented on IBM PC. The IMIS was developed to estimate the irrigation requirements and schedule for various types of crops during all growth stages for large number of farms in small and large irrigation projects under the local prevailing conditions of climate, water, soil and plants. The irrigation demands are reduced by considering the contributions of shallow water table and rainfall to the crop water requirements (ET crop). The IMIS reduces the quantities of possible water supply shortages from the irrigation water doses of the cultivated crops and distributes the reduced values between crops on the basis of yield response factor (Ky) of each crop during the insensitive growth stages to avoid any serious damages to the plants, and to minimize the economic losses. This IMIS has been implemented successfully during the past three years in irrigation schemes containing large numbers of center-pivots or farms in the Kingdom. In Al-Hassa project, the irrigation demands were reduced by about 72 million m3/year, and the allocation of the available irrigation water has been more effective especially during the water shortages in summer. The application of this system in SHADCO project has resulted in saving about 30 million m3/year or an average value of 49% of the used water quantities during normal operation. Agricultural yield has also improved. This model can be implemented in other agricultural projects in arid zones.

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RÉSUMÉ

Un model informatise pour reduire les besoins en irrigation et la gestion efficace des resources en eau limitees

Dans les pays arides et severement arides, telle L'Arabie Seoudite, ou les nappes d'eau souterraines constituent la source majeure d'aprovisionement, la gestion efficace des eaux d'irrigation est essentielle pour maintenir la qualite et la productivite a long terme des nappes souterraines. L'amelioration de la gestion des eaux d'irrigation comprend entre autre, le development et la realization de procedures nouvelles pour reduire les besoins en irrigation, et l'allocation rationelle de l'eau disponible entre les cultures envisagees la ou les disponibilites en eau sont limites.

Un programme infomatise nome systeme d'information et de gestion de l'irrigation (IMIS) a ete developpe et mis en application sur un IBM PC . IMIS permet d'evaluer les besoins en irrigation et le programme pour different type de cultures, pendant toute les etapes de leur croissance et ce pour un grand nombre de fermes impliques dans des projets d'irrigation, grand ou petit, prenant en consideration les conditions locales de climat, de disponibilite en eau, du sol et des plantes. Les demandes en irrigation sont reduites dans la mesure ou l'on prend en considerarion l'apport d'une part des nappes souterraines peux profondes et d'autre part des eaux de pluie.

IMIS permet d'eviter les possibilites de reduction des rations en eau d'irrigation des cultures. La reduction est distribuee entre plusieures cultures sur la base du "yield response factor" (Ky) de chaque culture durant la periode de croissance la moins sensible pour eviter tout domage majeure a la plante et reduire les pertes sur le plan economique. IMIS a ete utilise, avec succes pendant les trois dernieres annees, sur des projets d'irrigation comprenant plusieures grandes fermes centrales dans le royaume. Dans le cadre du projet d'Al-Hassa la demande en eau d'irrigation a ete reduite par environ 72 million de m^3/an et la distribution de l'eau d'irrigation disponible a ete plus efficace surtout pendant la periode ou la disponibilite est reduite c'est a dire en ete. L'utilisation d'IMIS sur le projet de SHADCO a permis d'economiser pres de 30 million de m^3/an soit en moyenne 49% de la quantite d'eau utilisee auparavant et ce avec une amelioration de la rentabilite a l'hectare. Ce programme peut etre utilise pour des projets agricoles en d'autre zones arides.

1. INTRODUCTION

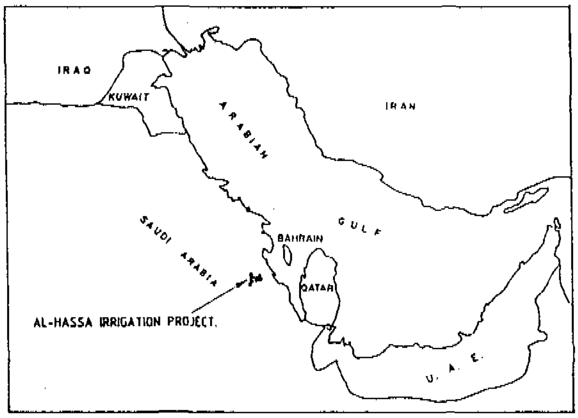
The irrigated areas in the world has increased from about 170 million ha in 1970 to about 240 million ha in 1990, to meet the growing food demands (Pallas, 1993). As a result, the irrigation water demands has increased drastically. In most developing countries the agricultural demands represents more than 80% of the total water demands (Hennessy, 1993). Unfortunately, the population growth in several parts of arid regions such as the Middle East is high (2.5%), while the low per capita water supply of 1000 m3/year will drop further to half of its present value in just 26 years (Jensen, 1993). This shows that in order to meet future increase in food demands with limited water supplies, improvement of water management using advanced technologies are very important to reduce the irrigation water consumptions, and to maximize the yield from each consumed water unit.

Saudi Arabia extends mostly in arid and severely arid regions. The irrigation water demands in the Kingdom has increased to about 19.6 billion m3 in 1994 (Dabbagh and Abderrahman, 1994) due to the expansion of irrigated areas from 0.5 million ha to 1.35 million ha during the same period. The irrigation water consumption represents more than 85% of the national water use. The groundwater resources is the source of about 94% of the national irrigation water consumption. With the support and encouragement from the government, large irrigation schemes containing thousands of hectares, and large number of farms or center pivot sprinkler irrigation systems were put under operation. Examples of the large irrigation schemes are Al-Hassa Irrigation and Drainage Project and Ash-Sharqiyah Agricultural Development Company (SHADCO) in the Eastern Province of Saudi Arabia (figures 1 and2).

The Al-Hassa project irrigates about 7,100 ha comprising about 21,000 farms cultivated with vegetables, date palm, alfalfa and cereals. The project consists of 19 main canal systems which bifurcate into submains and laterals. The irrigation system includes about 1650 Km of open concrete canals of trapezoidal, rectangular, and parabolic types with different sizes. The farmer gets his share of the flowing spring water on pre-determined dates from the lateral canals using siphoning method. Possible water supply shortages of more than 30% occurs in some canals during the peak demands in summer season. Shallow groundwater table exists at less than 3m below ground level in the whole project area.

SHADCO project irrigates about 5200 hectares using 67 center-pivot sprinkler irrigation systems and 67 deep wells (Figure 2). The main cultivated crops are wheat, barely and alfalfa.

Improvement of irrigation management in the Kingdom is essential for efficient use of the available water resources. This is important for satisfying the increasing water demands with the required qualities and quantities; and for maintaining the long-term productivity and quality of groundwater. The improvement of irrigation water management includes among several measures, the development of new techniques to reduce the irrigation demands by consideration of contribution from rainfall and shallow water table to the crop irrigation requirements; and the effective allocation of the available water among the cultivated crops under possible water supply shortages.



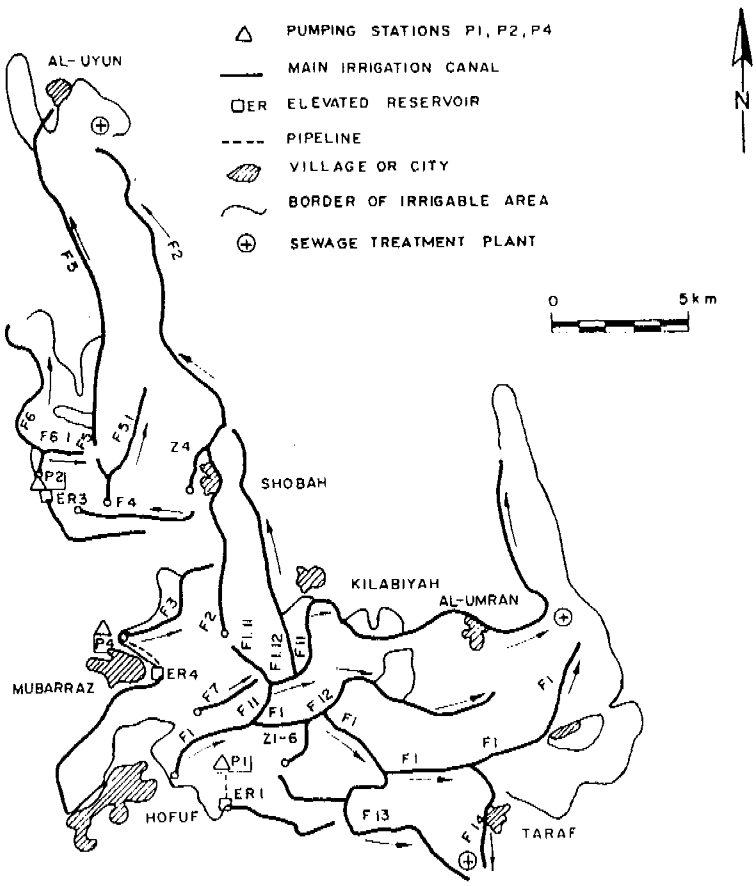


Figure 1 General map of Al-Hassa irrigation Project and the Main Irrigation System.

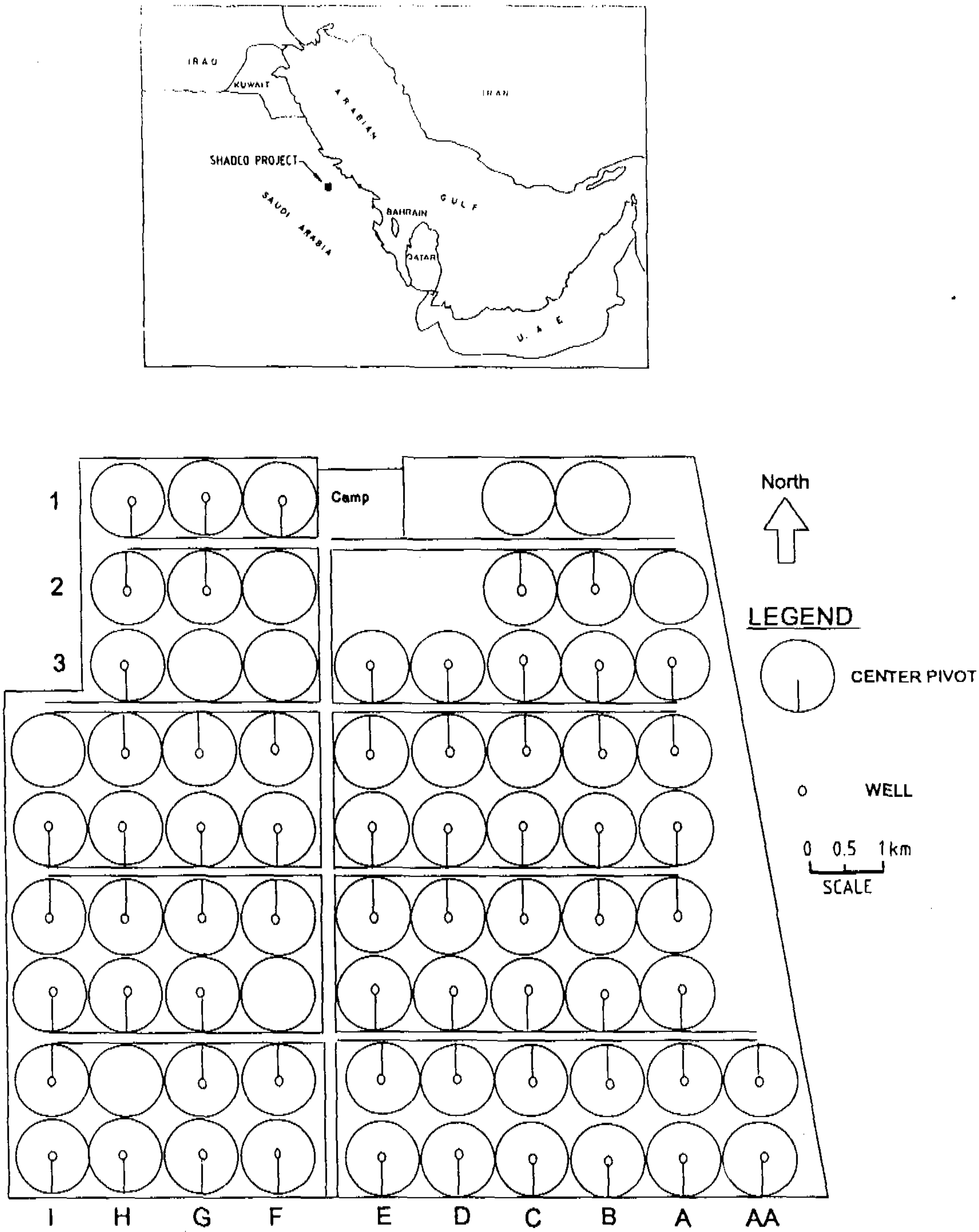


Figure 2. Map of Shadco project

Several computerized models were developed during the last two decades to simulate the process related to water transfer in soil-plant-atmosphere systems and for estimating irrigation depth and timing. A Computerized Irrigation Scheduling Service (CISS) based on calculations of water consumption and field measurements was developed and used in some Western States of the U.S.A (USDI, 1979). The Irrigation Water Requirement System (IWRS) was developed for use with full water supply to obtain maximum yield from irrigated crops (TWDB, 1975). The California Irrigation Management Information System (CIMIS) was developed to provide computerized irrigation scheduling programs for California on the basis of historical and real-time weather data and to improve water budget scheduling technologies (Snyder et al, 1985). RELREG, a model for real time irrigation scheduling was developed by Teixeira et al, 1993, to support farmer's decision on the irrigation doses and timings. But none of these models has been tested successfully for use in scheduling the water delivery for each type of crop at the farm level in complex irrigation project containing thousands of farms under possible water shortages and shallow groundwater conditions in severely arid conditions, such as that of Saudi Arabia.

A computerized Irrigation Management Information System (IMIS) has been developed and implemented on IBM PC. The program provides a sound water scheduling policy with possible water supply shortages during the peak irrigation demands in the hot summer season. It also reduces the irrigation water demands by effective account of the contributions from rainfall and shallow water table to the crop water requirements. The IMIS was developed to suit the large and smell irrigation projects containing large number of farms or center pivots under the local prevailing conditions of climate, water, soil and plants in Saudi Arabia.

This paper describes the developed IMIS and its application and benefits in the two large irrigation schemes in Saudi Arabia; and its future application with limited water supplies in arid regions.

2. DEVELOPMENT OF THE IRRIGATION MANAGEMENT INFORMATION SYSTEM(IMIS)

The development of the Irrigation Management Information System (IMIS) was based on the estimation of the following parameters:

1. Reference crop evapotranspiration (ETo).
2. Crop water requirements (ETcrop).
3. Shallow groundwater contribution to the ETcrop.
4. Deduction of the water shortages from Crop irrigation requirements on the basis of yield response factors of crops without causing serious damages to any plant.
5. Irrigation scheduling considering the reduced water doses.

The values of the reference crop evapotranspiration (ETo), and crop water requirements (ETcrop) are calculated using FAO recommended methods (Doorenbos and Pruitt, 1984).

Penman and Modified Penman methods were found to be the best to predict ETo values by the IMIS under Al-Hassa and SHADCO project conditions when compared with the measured ETo values (KFUPM, 1991).

The crop irrigation requirements (IR) is calculated from the following relationship (Doorenbos and Pruitt, 1984):

$$IR = (ETcrop - (Pe + Ge + Wb))/((1-LR)Ep) \qquad (4)$$

where:

IR = crop irrigation requirements in mm/day
ETcrop = crop evapotranspiration in mm/day
Pe = contribution from rainfall in mm/day
Ge = contribution from shallow groundwater in mm/day
Wb = contribution from carry-over of soil water in mm/day
LR = leaching requirements, fraction
Ep = irrigation efficiency of the project, fraction

Research studies under Al-Hassa conditions (HARC, 1976), and in several parts of the world (Grismer and Gates, 1988), (Ayers and McWhorter, 1985), (Alvino and Zerbi, 1986), and others found significant contribution from shallow groundwater table (Ge) to a depth of 3 meters to the ETcrop of several salt tolerant crops such as cotton, alfalfa, sorghum and barely. Ge is determined by its depth below the ground surface, the crop type, salinity of the groundwater and the soil texture. The suggested contribution of shallow groundwater table to ETcrop of alfalfa and date palm by HARC (1976) under Al-Hassa conditions are given in table (1).

The contribution from carry-over soil water (Wb) is calculated from the soil water at the beginning of each irrigation period. The measured values of this parameter were very low during the peak irrigation demands and with limited water supply such as Al-Hassa.
The calculated values of LR for the local varieties of cultivated crops (alfalfa, date palm, cereals, and vegetables) ranged from 0.14 to 0.25 in SHADCO project and Al-Hassa respectively. These values were ignored in IMIS because they were compensated by the

irrigation efficiency which is generally less than 0.65 in Al-Hassa (using surface irrigation) and 0.80 in SHADCO project (using center pivot sprinkler irrigation systems), as a part of the percolated water losses (KFUPM, 1991).

Table 1. The Contribution of moderately saline shallow groundwater table to the ETcrop of alfalfa and date palms.

Crop Type	Depth to Shallow Groundwater Table (cm)	Ge as % of ETcrop
Alfalfa	60-150	20
	150-280	15
Date palm	60-150	30
	150-300	25

The irrigation schedule identifies the depth of irrigation dose (d) and the interval between irrigation doses (i) according to the plant-soil-water relationship. The irrigation schedule is calculated using the recommendations of Doorenbos and Pruitt,(1984).

Measures to compensate for the water supply shortages are defined for each type of crops on the bases of yield reduction value. The yield reduction of any crop is dependent on the yield response factor (Ky) of that crop to water stress (Doorenbos and Kassam, 1985). The values of water shortages are deducted from the irrigation doses of the cultivated crops during the insensitive growth stages. The irrigation operator has the choice to select the deduction level from the irrigation doses of crops on the basis of their Ky values, and without causing any serious damages to the crops. The operator takes into account the economic value of various yield reductions alternatives of crops according to the local market, and selects the reduction combination with the least economic losses.

3. DESCRIPTION OF THE IRRIGATION MANAGEMENT INFORMATION SYSTEM (IMIS)

Two versions of the IMIS were developed. The first version, IMIS, was developed for irrigation schemes using multi branched open canal systems irrigating large number of farms. The second version, CIWMS, was developed for schemes using center pivot sprinkler irrigation systems. The IMIS consists of main computer program and fourteen different subroutines (Figure 3); and the CIWMS consists of main computer program and seven subroutines (figure 4). Both systems were developed using FORTRAN 77 language. The two systems are compiled using Microsoft FORTRAN Compiler Version 4.0 and implemented on IBM PC/386. Options in the computer program are chosen at the execution time through the interactive mode, through a simple question /answer session.

3.1 Brief Explanation of Different Subroutines of IMIS

CSS: Reads the configuration of any selected part of the system and the related available soil moisture in five layers of the soil profile (30 cm thickness each). Data are read in the interactive mode during execution.

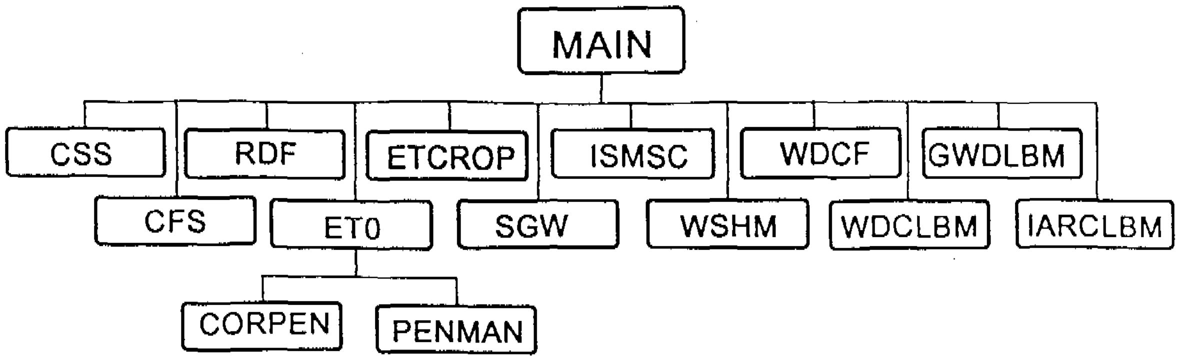


Figure 3. A tree diagram showing different subroutines of the Irrigation Management Information System (IMIS) for irrigation schemes using open canals.

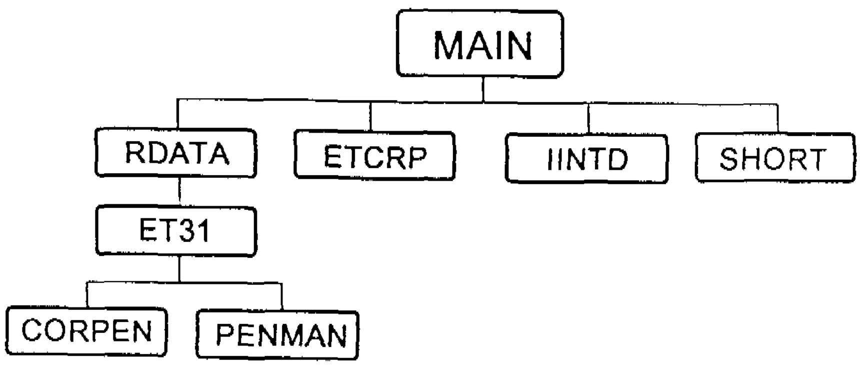


Figure 4. A tree diagram showing different subroutines of the Computerized Irrigation Water Management System (CIWMS) for irrigation schemes using center pivot sprinkler irrigation system.

CFS: Reads the configuration of the full system and the available soil moisture in five layers of the soil profile (30 cm thickness each).

RFD: Reads the farm related data.

ETCROP: Calculates the ETcrop.

SGW: Calculates the shallow groundwater table contributions to the ETcrop of alfalfa and date palm trees, and deducts them from the related ETcrop values of alfalfa and date palm.

ISMSC: Calculates the moisture storage capacity of the soil according to the root depth, the irrigation interval, net depth of irrigation application and gross depth of irrigation application considering the moisture extraction by plant roots from four soil quarters.

WDCF: Calculates the irrigation water demands by each crop in each farm in each lateral.

WDCLBM: Calculates the water demands by each crop group under each lateral, branch, and main irrigation system.

GWDLBM: Calculates the irrigation water demands by all crop types under each lateral, submain, and main system on monthly basis.

IARCLBM: Calculates the area of each crop group in each lateral, submain, and main system.

ET0: Reads all input data related to ETo calculation from real-time data, converts the data into metric units if needed, calculate the means, adjust the wind measurements to 2 meters height if the measured values are at another height, estimates the solar radiation if not given, and calls the subroutines PENMAN and CORPEN.

PENMAN: Calculates the ETo values using Penman method.

CORPEN: Interpolates the Penman correction factor "c", and calculates the corrected Penman value.

WSHM: Calculates the value of water shortages and reduces the irrigation doses of crops to compensate the water shortages according to the crop yield/water stress relations without causing serious damages to the plants.

3.2 Brief Explanation of Different Subroutines of CIWMS

RDATA: Reads the calculated ETo using historical data. It calculates the ETo values using real-time weather data, copmares them with the ETo values determined using historical data for the month, and adopts the ETo values using real time data if the difference between the two values is more than 10%. This subroutine calls subroutine ET31 to calculate the ETo from real-time weather data.

ET31: Reads all input data related to ETo calculation from real-time data, converts the data into metric units if needed, calculate the means, adjust the wind measurements to 2 meters height if the measured values are at another height, estimates the solar radiation if not given, and calls the subroutines PENMAN and CORPEN.

PENMAN: Calculates the ETo values using Penman method.

CORPEN: Interpolates the Penman correction factor "c", and calculates the corrected Penman value.

ETCRP: Calculates the ETcrop and the shallow groundwater table contributions to the ETcrop of alfalfa and date palm trees, and deducts them from the related ETcrop values of alfalfa and date palm..

IINTD: Calculates the moisture storage capacity of the soil according to the root depth, the irrigation interval and net depth of irrigation application considering the moisture extraction by plant roots from four soil quarters, gross depth of irrigation application, quantity of the irrigation dose and the required number of operation hours of the center pivot to be operated to deliver the calculated dose. It also deducts the values of the rainfall contributions from the irrigation dose.

SHORT: Calculates the value of water shortages and reduces the irrigation doses of crops to compensate the water shortages according to the yield response factor (Ky) of that crop to water stress without causing serious damages to the plants.

4. RESULTS OF SYSTEM FIELD APPLICATION

The IMIS was applied successfully during the past three years in two large irrigation schemes in the Eastern Province of the Kingdom. These schemes are Al-Hassa Irrigation and Drainage Project and SHADCO project.

In the Al-Hassa Project, the system was used to calculate the ETO, ETcrop and the net ETcrop values considering the contribution from shallow water table to ETcrop of date palm and alfalfa. The system calculates the irrigation demands for each crop, farm, lateral, submai and main canal, and the irrigation doses and intervals for the cultivated crops. The cultivated area with date palm in AL-Hassa Project is 4,684 ha (ignoring the mixed areas with vegetables and alfalfa); and the area of alfalfa is 1,384 ha. The use of IMIS has helped in reducing ETcrop of date palm and alfalfa by 25% and 15% respectively, by considering the minimal values of shallow water table contribution to ETcrop of the two crops. As a result, the total water demands of the cultivated date palm and alfalfa areas were reduced by about 72 million m3/year. It was also used to calculate the water shortages, and to deduct these values from the ETcrop of alfalfa and date palm according to the Ky values of the two crops, and avoiding any deduction during sensitive growth stage. Then it calculated the resulting irrigation doses and irrigation intervals for all types of crops in each farm, on each lateral, submain and main canal system. Better estimation and control on the distribution of water doses among large number of farms on daily and monthly bases was achieved especially with water shortages during summer months. The distribution of the available irrigation water has been more effective and just especially during the water shortages. Examples of the output of previous parameters are shown in tables (2-6). With the use of the computerized IMIS by the water operators, it was possible to calculate and to update the water operation schedule within a very short time for any number of farms, fields or irrigation canals with limited water supplies. This was difficult or impossible to be achieved without the IMIS.

The CIWMS was implemented in SHADCO Project during the last three years to calculate the ETo, ETcrop, Irrigation demands, irrigation doses and intervals and the operation times and durations of the center pivot sprinkler systems during the whole growth stages of wheat, barely and alfalfa. Table (7) shows a sample of the output of an irrigation operation schedule for wheat at SHADCO Project. The rainfall contribution was deducted from the irrigation doses of the crops during the rainy days. The application of the CIWMS at SHADCO Project has resulted in saving about 30 million m3/year or about 45% of the water quantities during normal operation. Moreover, the costs of operation and Maintenance were reduced, nearly in the same proportion. The agricultural yield in SHADCO project has also improved during 1991-1992 cropping season by more than 30% due among other reasons to the application of effective water operation system. The use of IMIS on PC made it possible and precise to simulate beforehand the water operation schedule during the whole growth stages of each cultivated crop in the beginning of the cropping season.

The application of the IMIS and CIWMS in the two projects has resulted in improving the water use efficiency and the water management. This has resulted significantly in conserving the long term productivity and quality of groundwater resources in two areas of the projects. Field measurements of piezometric water levels in the site of SHADCO project and groundwater simulation studies [Al-Assar, 1992] showed that the application of the CIMIS has resulted in minimizing the long term drawdown to less than 15cm/year instead of 60cm/year with normal water operation. This shows the potential benefits from the use of IMIS in severely arid regions to operate the irrigation projects with limited water supplies to conserve the water resources.

Table 2. Net Crop Evapotranspiration ET crop in mm/day for Alfalfa and Date palm after Considering the Groundwater Contribution

Month	Alfalfa	Date palm
MONTH= 1	2.85	2.18
MONTH= 2	4.02	3.47
MONTH= 3	5.24	4.72
MONTH= 4	6.50	6.06
MONTH= 5	8.90	7.07
MONTH= 6	9.80	7.86
MONTH= 7	10.31	8.05
MONTH= 8	8.44	7.86
MONTH= 9	6.62	5.60
MONTH= 10	5.10	3.94
MONTH= 11	3.55	3.06
MONTH= 12	2.75	2.09
Average	6.17	5.12

Table 3. The Calculated Percent Yield Reduction in Alfalfa, and Date Palms at Different Levels of Water Shortages for F4 Irrigation System

MONTH	CROP	REDUCTION %	YIELD %	CROP	REDUCTION %	YIELD %
1	TREE	0.0	100.0	ALFALFA	0.0	100.0
2	TREE	0.0	100.0	ALFALFA	0.0	100.0
3	TREE	0.0	100.0	ALFALFA	0.0	100.0
4	TREE	4.3	97.2	ALFALFA	4.6	98.6
5	TREE	12.8	91.7	ALFALFA	40.0	83.2
6	TREE	25.9	74.1	ALFALFA	40.0	83.2
7	TREE	19.4	80.6	ALFALFA	40.0	83.2
8	TREE	5.4	94.6	ALFALFA	40.0	83.2
9	TREE	0.0	100.0	ALFALFA	0.0	100.0
10	TREE	0.0	100.0	ALFALFA	0.0	100.0
11	TREE	0.0	100.0	ALFALFA	0.0	100.0
12	TREE	0.0	100.0	ALFALFA	0.0	100.0

Table 4. Irrigation schedule for alfalfa on lateral No. 1 in days.

MONTH=1	18
MONTH=2	13
MONTH=3	10
MONTH=4	8
MONTH=5	6
MONTH=6	5
MONTH=7	5
MONTH=8	6
MONTH=9	7
MONTH=10	10
MONTH=11	15
MONTH=12	18

Table 5. Irrigation demand for alfalfa at Farm, Lateral, Submain, and Main canal levels (m3/month).

SUBMAIN NO. = 5, LATERAL NO.= 2, FARM NO. = 1

ALFALFA

1	279.4	2	356.7	3	514.0	4	617.7
5	873.4	6	930.9	7	1011.7	8	828.4
9	629.3	10	500.2	11	337.2	12	270.1

SUBMAIN NO. = 1, LATERAL NO.= 3.

ALFALFA

1	5281.3	2	6471.5	3	9714.6	4	11675.2
5	16507.4	6	17593.7	7	19121.5	8	15657.4
9	11892.9	10	9453.8	11	6373.3	12	5104.0

SUBMAIN NO. = 1

ALFALFA

1	100340.2	2	128085.6	3	184572.2	4	221823.0
5	313632.0	6	334271.4	7	363298.7	8	297481.9
9	225957.9	10	179616.3	11	121089.3	12	96972.3

MAIN IRRIGATION SYSTEM

1	197099.6	2	251600.3	3	362557.8	4	435730.1
5	616071.7	6	656613.9	7	713632.7	8	584347.9
9	443852.3	10	352822.9	11	237857.4	12	190484.1

Table 6. Total irrigation demand at lateral, submain, and main irrigation system levels (m3/month).

SUBMAIN NO. = 1, LATERAL NO.= 1.							
1	13470.0	2	19319.2	3	29141.8	4	36147.5
5	43620.2	6	46913.4	7	49633.6	8	45470.5
9	33398.9	10	24298.7	11	18264.0	12	12877.9
SUBMAIN NO. = 1							
1	490338.6	2	722126.3	3	1097985.0	4	1399190.0
5	1730586.0	6	1790803.0	7	1813017.0	8	1694874.0
9	1334007.0	10	1011425.0	11	714498.3	12	469687.9
MAIN IRRIGATION SYSTEM							
1	1046620.0	2	1513028.0	3	2368776.0	4	2978464.0
5	3691578.0	6	3848651.0	7	3981802.0	8	3668564.0
9	2838732.0	10	2110592.0	11	1472712.0	12	1002282.0

5. CONCLUSIONS

An irrigation management information system (IMIS) was developed and implemented successfully in two large irrigation systems in Saudi Arabia under extremely arid climate, and with shallow water table and possible water shortages. The implementation of the IMIS on the Al-Hassa Project, which uses the open canal systems, resulted in reducing the irrigation water demands by about 72 million m3/year or a saving of about 30%; and in solving the problems of water allocation under water supply shortages. The implementation of the other version CIWMS on SHADCO Project has resulted in saving about 45% of the irrigation water use during normal operation and contributed to increasing the yield by more than 30%. This is due to effective account for the actual irrigation requirements and the contribution of the rainfall to the crop irrigation requirements during all growth stages. The application of the developed systems have resulted in substantial benefits on irrigation water management and groundwater conservation. The costs of operation and maintenance were reduced significantly and the functional life of pumps and irrigation equipments is preserved. The IMIS can be applied effectively on any irrigation scheme in the GCC countries and in similar arid regions of the world.

Table 7. Sample of the output of an irrigation operation schedule for wheat at SHADCO Project.

COMPUTERIZED IRRIGATION WATER MANAGEMENT SYSTEM
(CIWMS)
FOR
ASH-SHARQIYAH AGRICULTURAL DEVELOPMENT COMPANY
(SHADCO)
DEVELOPED BY
WATER RESOURCES AND ENVIRONMENT DIVISION
THE RESEARCH INSTITUTE
KING FAHD UNIVERSITY OF PETROLEUM AND MINERALS
DHAHRAN 31261
SAUDI ARABIA

RABI' II 1412 - OCTOBER 1991

FIELD F4 crop wheat STAGE NO 5 BEGINNING OF STAGE 56 END OF STAGE 115
CROP COEFFICIENT 1.100

DAYS	DATE FROM	DATE TO	ROOTING DEPTH CM	IRRIGATION INTERVAL HOURS	APPLI DEPTH MM	QUANTITY CU.MTS	OPERATI HOURS
56- 62	16/ 2/93	- 22/ 2/93	38.00	17.40	7.63	6488.84	17.40
63- 68	23/ 2/93	- 28/ 2/93	40.00	18.32	8.04	6830.36	18.32
69- 69	1/ 3/93	- 1/ 3/93	40.00	13.79	8.04	6830.36	13.79
70- 76	2/ 3/93	- 8/ 3/93	45.00	15.51	9.04	7684.15	15.51
77- 83	9/ 3/93	- 15/ 3/93	50.00	17.24	10.04	8537.95	17.24
84- 90	16/ 3/93	- 22/ 3/93	50.00	17.24	10.04	8537.95	17.24
91- 97	23/ 3/93	- 29/ 3/93	50.00	17.24	10.04	8537.95	17.24
98- 99	30/ 3/93	- 31/ 3/93	50.00	17.24	10.04	8537.95	17.24
100-104	1/ 4/93	- 5/ 4/93	50.00	14.12	10.04	8537.95	14.12
105-111	6/ 4/93	- 12/ 4/93	50.00	14.12	10.04	8537.95	14.12
112-115	13/ 4/93	- 16/ 4/93	50.00	14.12	10.04	8537.95	14.12

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EVALUATION OF OPTIMAL STOCHASTIC OPERATIONAL POLICIES FOR THE HIGH ASWAN DAM

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ABSTRACT

A stochastic dynamic programming model, referred to as the Steady State Model, was developed by researchers at the Massachusetts Institute of Technology for the Egyptian Ministry of Public Works and Water Resources to provide guidance on optimal release policies for the High Aswan Dam. The stochastic dynamic programming model utilizes probability transition matrices for stochastic modeling of Lake Nasser inflows as a lag-1 Markov process. With the neglecting of cost discounting, the Steady State Model should produce hedging policies which are essentially independent of the initial system state (i.e., reservoir storage in Lake Nasser). Surprisingly, the Steady State model was found to produce reservoir operational policies that behave much like the high discount case, which intensify the risk of incurring severe shortages during extreme drought periods. For comparison purposes, the generalized dynamic programming package CSUDP was applied, which utilizes an expected total cost criterion with discounting usually included. The CSUDP Model was successfully run without the use of discount factors and, as expected, produced hedging type policies, which reduce the risk of severe shortages. Test runs were made to assess the ability of the Steady State Model to produce policies that reflect varying levels of risk that the reservoir operators might be willing to take in violating the energy and irrigation targets. It was concluded that the model makes no provision for development of risk sensitive policies. The CSUDP model includes the capability of specifying risk parameters that allow certain constraints to be violated at a prespecified risk level.

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1. INTRODUCTION

The High Aswan Dam is the most dominant regulatory facility in the Nile Basin and of singular importance to the economy and life of Egypt (Figure 1). The High Dam was completed in 1970 and impounds Lake Nasser, which is one of the largest manmade lakes in the world. The High Dam provides controlled releases that enable perennial irrigation in the Nile Valley. The High Dam also provides the great majority of hydroelectric power production on the Nile, as we ll as serving important purposes in flood control and river bank stabilization. The development and implementation of optimal operational strategies is essential to maximizing the benefits of the High Dam to Egypt.

The Stochastic Steady State operational model, based on stochastic dynamic programming, has been developed for optimal operation of the High Dam. A comprehensive evaluation of this model is presented herein, including a review of the underlying theoretical concepts and an analysis of the implementation of those concepts into the software. Further evaluation is provided through comparison of results with those obtained from the generalized dynamic programming package CSUDP developed at Colorado State University. The evaluation addresses the ability of the Steady State Model to yield risk-sensitive operational strategies by evaluating the long-term performance of the policies using the Lake Nasser Simulation Program. An alternative risk evaluation procedure is described which includes incorporation of conditional risk targets directly into the CSUDP optimization. It is believed that these studies provide the basis for implementation of optimal operating rules obtained from these models, with consideration of the risks associated with failing to satisfy desired operational targets and constraints.

2. STRUCTURE OF THE STEADY STATE MODEL

The Steady State Model for the High Aswan Dam was originally developed by El Assiouti, et al. (1979) and Alarcon and Marks (1979) through research jointly conducted at the Massachusetts Institute of Technology and Cairo University for the Ministry of Public Works and Water Resources in Egypt. Adaption of the model to operational planning studies for the High Aswan Dam has been variously supported through grants from the U.S. Agency for International Development, the World Bank and UNDP. The most recent report on the model is contained in a report from the Ministry of Public Works and Water Resources (1988), which includes description of an interactive program for convenient user-interfacing with the Model.

2.1 High Aswan Dam Operational Objectives and Constraints

The Steady State Model attempts to represent operations of the High Aswan Dam as a seasonally stationary Markov decision process. The objective is to develop optimal reservoir release policies that minimize the expected *weighted* squared deviation from operational targets on water supply and hydropower production, subject to satisfying reservoir (level pool) mass balance and various physical, legal and institutional constraints on operation of the system. The state of the system is defined by two variables: (i) reservoir storage volume at the beginning of the period and (ii) previous month inflows as a means of considering the lag-1 autocorrelation of monthly inflows. Transition probabilities are used for representing inflow state persistence over succeeding months. They are defined as the probability of occurrence of a specific discrete inflow class during a particular month, conditioned on inflow magnitudes within a specific inflow class during the previous month. Transition probability matrices are estimated within the Steady State Modeling Package. Time series of *naturalized* inflows to Aswan Dam are input and the associated monthly means, standard deviations, lag one auto-correlation coefficients and transition probability matrices calculated. Naturalized inflows should be used since they take into account upstream storage effects on the flows.

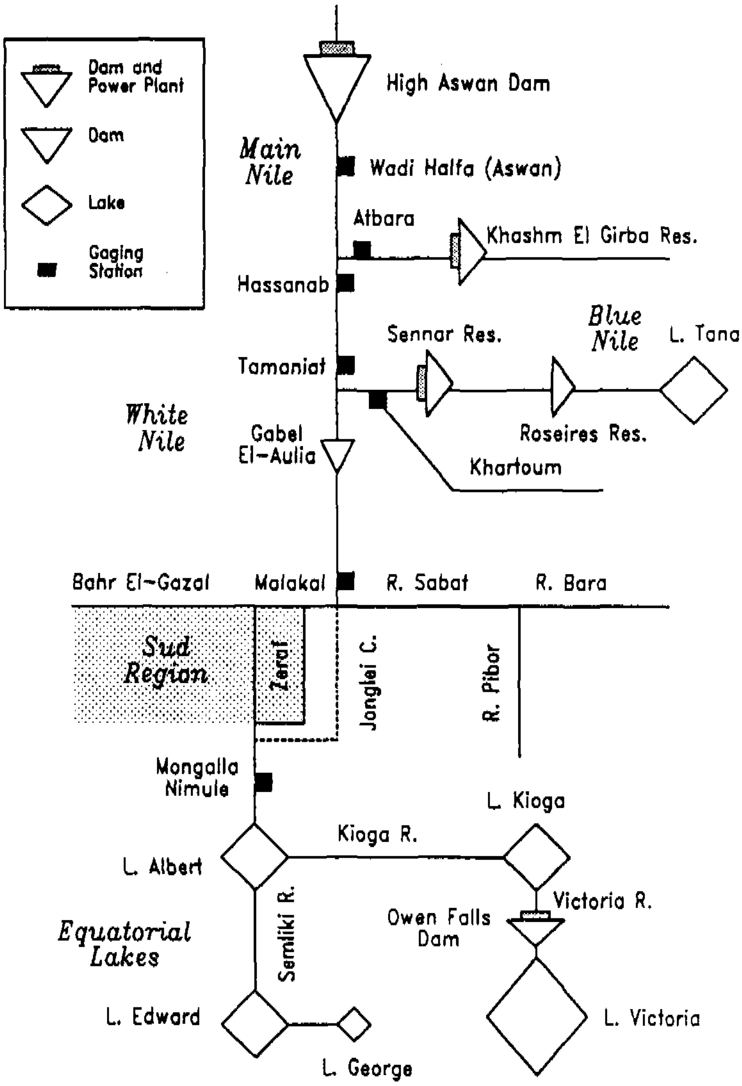

Figure 1. Scematic representation of the Nile Basin

Operation rules used for the High Aswan Dam are applied in accordance with the Nile Agreement between Egypt and Sudan which establishes amounts of water to be shared between the two countries. Historical operation rules have required that full downstream demands be released during periods of normal or near-normal flows (i.e., when river flows are near the average annual yield of 84 billion cubic meters (BCM)). Constraints are imposed on maximum downstream releases to avoid channel degradation. In addition, the Ministry of Public Works and Water Resources attempts to control the annual flood by drawing down the reservoir to a predetermined level by July 31 of each year. A succession of several low flow years results in institution of a *shortage rule* or *sliding scale* as a function of reservoir storage which imposes increasing shortages on downstream demands as reservoir storage approaches dangerously low levels.

The current operating rules do not adequately account for the stochastic nature of Nile River inflows and fail to incorporate the risks associated with water supply and energy production shortages. It is believed that implementation of the dynamic programming policies can produce significant improvements over the present operational policy by better integrating operations for hydropower production and irrigation release, and quantifying shortage rules to imposed during periods of low flow and drought.

2.2 Operational Objective Function

The objective function employed in the Steady State Model specifies penalty functions for each period associated with expected present and fu ture violation of desired operating targets on release for downstream water supply and hydropower generation. An additional penalty term is employed in the objective function to discourage high releases that can result in degradation of the river channel downstream.

The objective function for both models for any month t is of the form:

$$f_t(S_t,S_{t+1},R_t) = a_1\,(TI_t-R_t)^{b_1} + a_2\,(TE_t-E_t(S_t,S_{t+1},R_t))^{b_2} + a_3[\max\,(0,R_{\max}-R_t)]^{b_3} \tag{1}$$

where S_t is discrete reservoir sto rage at the beginning of period t, R_t is the total volume of release during period t, TI_t is ideal target release for irrigation, $E_t(S_t,S_{t+1},R_t)$ is actual hydropower generation during period t as a function of average head on the turbines over period t, which is in turn a function of beginning and ending period storage levels S_t, S_{t+1}, respectively; TE_t is ideal target hydropower energy generation during period t; $R_{\max}$ is maximum desired release to avoid downstream channel degradation; and a_i,b_i $(i=1,2,3)$ are coefficients associated with the penalty terms. All volume units are expressed in milliard cubic meters or billion cubic meters (BCM) and energy production is in gigawatt-hours (GWH). Some sensitivity analyses have been previously performed on these coefficients, but normally the b_i are set to 2, and the a_i $(i=1,2,3)$ are set at 10^4, 10^{-3}, and 10^4, respectively.

Irrigation release targets TI_t are based on an estimated annual downstream demand of 55.5 BCM, which is distributed according to prespecified percentages. The energy target TE_t, on the other hand, is set to an unattainably high value (i.e., 1750 GWH/month) to encourage the optimization model to maximize energy production, while maintaining irrigation requirements according to the weighting coefficients specified.

2.3 Reservoir Mass Balance

End-of-period storage is calculated from conservation of mass for the reservoir as:

$$S_{t+1} = S_t - R_t + Q_t - L_t(S_t, S_{t+1}) \tag{2}$$

Notice that equation 2 is an implicit function of S_{t+1}, which means that iterative calculations must be employed for determining S_{t+1}, given S_t, R_t. Losses $L_t(S_t, S_{t+1})$ are estimated during period t as primarily due to evaporation, seepage, and possible spills at Toshka. Losses are shown as functions of beginning and ending storage since evaporation is a function of average surface area over period t.

Annual evaporation depth is assumed to be 2.874 m and is distributed into monthly amounts according to specified percentages. Total evaporation is calculated as evaporation depth for a particular month times average surface area over that month. Surface area is calculated as

$$A(S) = e^{(0.8149 \cdot \ln S + 4.63844)} + 78.2 + S \cdot (-3.3333 + S \cdot (0.0394 - S \cdot 1.337 \times 10^{-4})) \tag{3}$$

It should be noted that this equation has recently been modified by the Ministry of Public Works and Water Resources. These changes were not incorporated in this present study so that comparisons could be made between CSUDP Model results and published results of the Steady State Model, which are based on the previous relations.

Inflows Q_t are *naturalized*, or adjusted for upstream regulation, as well as reduced by the amount of Sudan abstractions. Sudan abstractions total 14.8 BCM annually, and are subtracted monthly according to predetermined.

Toshka is an uncontrolled spillway constructed on the west side of the reservoir which is designed to spill to the desert at elevation 178 m in order to avoid excess downstream releases that can cause degradation problems. The daily discharge in MCM is calculated as

$$Q_{TOSH} = \exp\left(-0.6077 + 1.809\, e^{(H-178)} - 0.073\left(e^{(H-178)^2}\right)\right) \quad \text{if } H > 178\, m \tag{4}$$

Toshka spills are approximated as the average of spills at the beginning and end of the period. Since calculation of the spills can in turn affect the elevations in the reservoir, an iterative process is used to adjust the heads and recalculate spills until convergence occurs. In fact, it appears that this is an inaccurate means of calculating Toshka spills and an improved procedure is required. The reason is that if a head of 178 m is attained sometime during a particular month, but head at the beginning of the month was less than 178 m, then the Toshka spill will be overestimated with this procedure. A better procedure would be to first estimate using linear interpolation the day within the month that a head of 178 m is attained. Toshka spill should then be calculated from that day to the end of the month, assuming that head at the end of the month exceeds 178 m. The same iterative procedure can be used for adjusting heads as a result of the spills occurring.

2.4 Operational Constraints

Bounds on storage (in BCM) and releases (in BCM/month) are, respectively:

$$\begin{aligned} 37.2\ [@147\,m] \le S'_{t+1} \le 168.9\ \ [@183\,m] \\ 3.2 \le R_t \le 7.6 \end{aligned} \tag{5}$$

The minimum storage level specifies an inactive or *dead* zone which is allocated to receive sediment trapped by the reservoir; hence its available volume is expected to decrease over time as sediment continues to accumulate. The level of this zone is established at 147 m. If the reservoir level is at 147 m or below, then no water can be released. Figure 2 depicts the various operating zones of the High Dam.

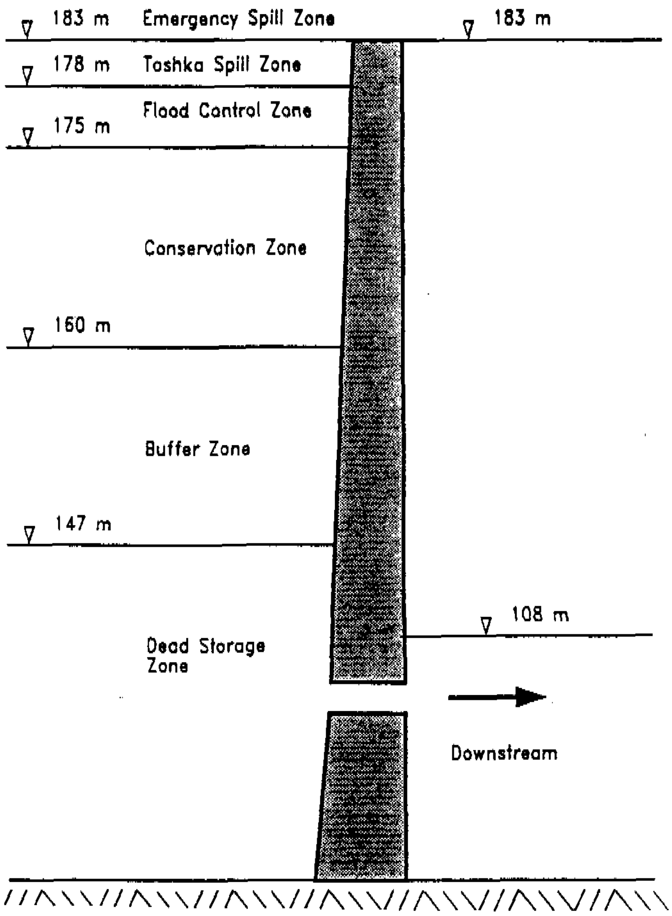

Figure 2. Schematic of storage zones at the High Aswan Dam

Requirements for controlling downstream channel degradation imposes an upper limit on release R_{max} which is estimated to be 7.6 BCM/month, corresponding to a maximum daily release of 250 million cubic meters. This is not set as an absolute constraint in the model, but is indirectly imposed through use of the penalty term in equation 1. A lower limit on release is established in order to sustain navigation. The navigation requirement is assumed to be satisfied at a minimum release of 3.2 BCM/month. The dynamic programming optimization is performed with strict adherence to upper and lower limits on storage capacity in the reservoir. The model appropriately adjusts releases to avoid violating these bounds.

2.5 Hydropower Generation

There are 12 Francis type turbines at the Aswan power plant with 175 MW power capacity each, for a total capacity of 2100 MW. However, the dam maintenance program requires that two turbines be out of service at any given time. The operational characteristics of the turbines as a function of discharge, head and expected power generated are given in Technical Report 22 (UNDP/EGY/81/031). These curves are incorporated into the Steady State Model and a linear interpolation scheme is employed for calculating hydropower energy generation $E_t(S_t,S_{t+1},R_t)$. These Calculations include functions relating the maximum turbine release and hydropower generation possible at a given head. The equation relating head on the power plant to reservoir storage is given by

$$H(S) = 26.444 \cdot \ln(S+16) + 44.063 + S\cdot(0.04243 + S\cdot(-6.689\times10^{-4} + S\cdot(4.088\times10^{-6} - S\cdot 8.4235\times10^{-9}))) \tag{6}$$

Again, this equation has been recently modified by the Ministry of Public Works and Water Resources, but its used is continued here to provide a basis for model comparison with published results.

Tables are input relating maximum release through the turbines as a function of head $R_{turb}(H)$ Tables are also produced which provide the maximum energy generation possible as a function of head $E_{turb}(H)$. The minimum possible release $R_{\min}(H)$ required for energy production and the minimum possible energy production $E_{\min}(H)$ are also input as tabular functions of head H . The slope of the energy-release curve is also input as a tabular function of head and total energy generation is then calculated as:

$$E_t(S_t,S_{t+1},R_t) = E_{slope}(\overline{H})\cdot(R_t - R_{\min}(\overline{H}) + E_{\min}(\overline{H}) + 0.73\cdot(7.58 + 2.6172 \cdot(1000\,R_t/30.4) - 0.00296\cdot(1000\,R_t/30.4)^2) \tag{7}$$

where $\overline{H} \equiv \overline{H}(S_t,S_{t+1})$ represents average head over period t as a function of the beginning and ending period storage. A linear interpolation scheme is used for selecting intermediate values from the tabular data.

In the coding of the model, a power bypass or spill term is calculated for releases above the maximum capacity of the turbines. However, it can be seen in the above equation that *total* release is actually used for producing energy. Although this appears to be an error in the program, these calculations were not altered in order to compare CSUDP with published results from the Steady State model. Future work should carefully examine these relations to determine if they have been coded correctly by comparing computed energy production with actual measured generation.

2.6 Inflow Transition Probabilities

The Steady State Model utilizes lag-one transition probabilities to stochastically represent naturalized inflows to the High Aswan Dam. These are derived assuming flows in any month are normal or log-normally distributed. It is also assumed that any two consecutive months are multivariate normally distributed. Conditional lag-one probability density functions are then calculated by:

$$p(q_t|q_{t-1}) = \frac{p(q_t,q_{t-1})}{p(q_t)} \tag{8}$$

These distributions are then discretized into five inflow classes which are represented by the median inflow in each class. This results in discrete probability transition matrices $P[Q_t|Q_{t-1}]$ for each calendar month t.

The question arises as to whether the assumption of normal or log-normal distributions is a valid one. Although preliminary computational studies were performed using these previously calculated transition probabilities, additional runs were made by reevaluating the transition probabilities by performing a frequency analysis on the historical naturalized flows over the period 1913 to 1989.

2.7 Dynamic Programming Optimal Return Function

The backward stochastic dynamic programming optimal return function for the Steady State Model is of the following form:

$$F_n(S_t,Q_{t-1}) = \min_{R_t} \sum_{Q_t} P[Q_t|Q_{t-1}]\,[f_t(S_t,S_{t+1},R_t) + \beta F_{n+1}(S_{t+1},Q_t)]$$

$$(\textit{for } n = N, N-1,\ldots,1; \textit{ and } t=n-INT(n/12)\cdot 12; \quad \textit{where if } t=12, \textit{ then } t+1 = 1) \tag{9}$$

where t is the current calendar month, $\beta = 1/[(1+r)]$ for monthly discount rate r ; N is the total number of periods in the successive approximations procedure, Q_t is naturalized inflow during month t, and R_t is reservoir release during month t. Generally, $N = 12 \cdot M$, where M is the total number of successive approximation cycles in the stochastic dynamic programming calculations. Although the operational horizon is theoretically infinite, increasing finite values of M are used until operating policies converge to stationary values. It should be noted that formulation of the steady state model in

Buchanan and Bras (1981) specifies the number of stages *remaining* as the stage counter, but the solution procedure can still be regarded as a backwards recursion.

The boundary conditions initialize $F_{N+1}(\cdot)$ for all system states; therefore $F_n(S_t, Q_{t-1})$ represents the minimum total expected penalty cost over periods $n, n+1, \ldots, N$, assuming reservoir storage is at current discrete level S_t and previous month inflows were Q_{t-1}. Transition probabilities $P[Q_t|Q_{t-1}]$ represent the probability of discrete inflow event Q_t occurring during period t, conditioned on previous month inflows occurring within discrete class Q_{t-1}. Upon convergence, optimal periodically stationary release policies $R_t^*(S_t, Q_{t-1})$ are obtained which are conditioned on initial storage and previous month inflow for each calendar month t.

2.8 Coding of the Steady State Model

An extensive review of the FORTRAN coding of the Steady State Model has confirmed that the logic flow is correct for the most part. Questions about the energy calculations were alluded to previously. Certain other inaccuracies have been uncovered, however, and corrected. It was found that the model output was producing negative values for the objective function, which is mathematically impossible since the objective function is composed of penalty terms that employ squared-error or L_2 norms on meeting certain ideal target releases and energy output. The problem was eventually traced to incorrect coding related to normalization of the dynamic programming optimal return function for showing convergence of the successive approximations or value iteration process. It was found that a normalized optimal return function value from the previous stage, which can have a negative value, was being incorrectly added to the non-normalized objective function values in the current stage. This error can lead to development of nonoptimal policies, but was easily corrected in the model. Another error was noted in the interpolation scheme used for obtaining dynamic programming optimal return function values stored over a discrete state-space. In certain cases, values were being incorrectly extrapolated rather than interpolated. This defect was also corrected in the model.

A feature of the Steady State Model that is cause for some concern is that the optimal search process always begins with an initial release level above which degradation downstream of Aswan would occur (i.e., 7.6 BCM per month), and then cycles through successive reductions of this level. In fact, there is a penalty term in the objective function which indirectly discourages releases above the degradation level. It is conceivable that on rare occasions, releases slightly above the degradation level might be warranted, but would not be allowed with the current model structure. Also of concern is the use of incorrect reservoir mass balance calculations in the model for the specific case where both calculated reservoir storage levels and releases exceed specified maximum levels. The Steady State Model simply truncates both of these values to their respective maximum levels without adding the additional spill onto total spill from the reservoir. This error would only be important during extreme high flow conditions, which rarely occur.

Aside from these correctable errors in the Steady State Model, the most critical concern is that the algorithm as coded is actually performing a stochastic dynamic programming analysis based on what Dreyfus and Law (1977) refer to as an *average cost per period* criterion, rather than total cost as implied in the formulation. As shown by Ross (1983), this formulation involves replacing the dynamic programming optimal value function $F_n(S_t, Q_{t-1})$ with the following function:

$$H_n(S_t, Q_{t-1}) = F_n(S_t, Q_{t-1}) - F_n^{\beta}(S_t^0, Q_{t-1}^0) \tag{10}$$

where the function $F_n^{\beta}(S_t^0, Q_{t-1}^0)$ represents the optimal solution to equation 9 under discount rate β for some given nominal states S_t^0, Q_{t-1}^0. In the program, these are chosen to be mean storages and flows. As stated in Dreyfus and Law (1977), the average cost criterion "has many difficulties that the discounted case does not possess." In fact, this method is intended as an option to the discounted case, and therefore is used without a discount rate. Ross (1983) proposes that the average cost criterion problem be solved using a linear programming formulation rather than dynamic programming. Solution by dynamic programming requires that $F_n^{\beta}(S_t^0, Q_{t-1}^0)$ somehow be approximated at each stage, since its exact value would not be known until convergence occurs.

In summary, the stochastic dynamic programming formulation which is presented in Buchanan and Bras (1981) and later reports is different from what has actually been programmed in the software. However, the formulation as described in Alarcon and Marks (1979) correctly reflects the way the software is actually written. In fact, it would be more theoretically correct to follow the formulation as given by Buchanan and Bras (1981).

Since the Steady State Model is not a generalized model, but has been specifically designed for operational analysis of Aswan Dam, there are many assumptions and values which have been imbedded into the code which may be difficult for the user to detect since they are not part of the input data file. For example, the algorithm always fixes the discretization interval on releases at 0.2 BCM. Various convergence tolerance parameters are also imbedded within the code, such as automatically commencing the optimization with respect to the releases at a level 2.0 BCM above the previous optimal policy. Since the optimization proceeds from high storage levels and inflows to low levels, this would appear to be a reasonable approach.

3. COLORADO STATE UNIVERSITY DYNAMIC PROGRAMMING MODEL

As a further means of evaluating the Steady State Model, the CSUDP model developed at Colorado State University was also applied to operation of Aswan Dam. CSUDP is a generalized dynamic programming model and has been widely used and thoroughly tested. CSUDP has been extensively applied for reservoir operation problems, such as reported by Allen and Bridgeman (1986). A complete description of the model is available in Labadie (1990).

Until the advent of the CSUDP program, users of dynamic programming had been forced to perform the laborious task of developing new dynamic programming code for each new application. With CSUDP, the user need only define the specific features of the sequential decision problem to be solved. This includes a wide range of multidimensional and stochastic problems. The code is designed to completely separate the knowledge-base (i.e., system relations describing state transformations and objective functions), the data base, and the control or optimization algorithm. The user develops user-supplied subroutines describing the system state transformation relations (Subroutine STATE) and objective function (Subroutine OBJECT), which are then linked to the main program. The data base and subroutines are prepared through an interactive, menu-driven interfacing program. An additional user supplied subroutine (Subroutine READIN) allows the user to customize applications of CSUDP to specific problems such that technical details of the dynamic programming algorithm become transparent. Likewise, mechanisms are included to customize output from the model for specific applications.

For stochastic problems, CSUDP employs the total cost criterion rather than the average cost per period criterion used in the Steady State Model. It is believed that the total cost criterion is more theoretically correct for use with value iteration or successive approximation approaches using dynamic programming. The dynamic programming recursion relation is:

$$F_n(S_t, Q_{t-1}) = \min_{R_t} \sum_{Q_t} P\,[Q_t | Q_{t-1}]\ [\beta^{n-1} f_t(S_t, S_{t+1}, R_t) + F_{n+1}(S_{t+1}, Q_t)]$$

$$\begin{array}{l}(\mathit{for}\ \ n = N, N-1, \ldots, 1;\ \mathit{and}\ t = n - INT\,(n/12) \cdot 12; \\ \quad \mathit{where}\ \ \mathit{if}\ t = 12,\ \mathit{then}\ \ t+1 = 1)\end{array} \qquad (11)$$

It is proved by Ross (1983) that convergence to stationary policies is guaranteed with this algorithm as long as the state-space is a countable set. A discount rate need not be included and single stage costs are not required to be bounded. In this formulation, all costs are directly discounted back to stage 1. This is in contrast with the Steady State Model formulation, which discounts the dynamic programming optimal value function one stage at a time as the backwards calculations proceed.

CSUDP has an additional capability not included in the Steady State Model which allows conditional *risk* estimates associated with violating constraints to be directly incorporated in the optimization. For the stochastic case, bounds are allowed to be violated up to an acceptable level of risk or probability of failure as specified by the user. Storage S_{t+1} as calculated in equation 2 is actually a random variable, since inflows Q_t are random variables. Therefore, constraints on S_{t+1} should be defined probabilistically. Although the Steady State Model accounts for violation of S_{t+1} by appropriately adjusting releases, there is no provision for the fact that the adjusted releases may actually be in violation of their bounds. Thus, the Steady State Model makes no direct provision for risk of violating release constraints in its structure.

The CSUDP Model allows the user to specify probabilistic constraints on storage as:

$$\begin{array}{l} P\,[S_{t+1} < S_{\min}] \le \alpha_1 \\ P\,[S_{t+1} > S_{\max}] \le \alpha_2 \end{array} \qquad (12)$$

where α_1, α_2 are desired risk levels associated with violation of the lower and upper bounds on storage. It should be reiterated that these are conditional risk values which are dependent on the known initial state of the reservoir system.

An additional capability of the CSUDP Model which is not available in the Steady State Model is a *splicing* option available for optimization over discrete release decision variables. The Steady State Model is limited to a discretization interval of 0.2 BCM, which may be a coarser interval than desired. The CSUDP Model allows selection of an initial coarse interval, and then performs a successive splicing around optimal solutions under the current discretization interval by confining the solution to increasingly refined corridors. The user may specify any final accuracy for the discretization interval and obtain this solution in a modest amount of computer time, as opposed to initially selecting a highly refined discretization interval. The user is provided options on the rate of splicing and the size of the corridor selected around current solutions.

An additional useful feature of the CSUDP Model is provision for a *tie breaking* option which allows users to determine whether optimal solutions are unique. The user is allowed to specify whether the *first* or *last* tie value is retained as optimization is progressing. This option is also not available in the Steady State Model.

4. COMPARATIVE EVALUATION OF STEADY STATE MODEL

4.1 Data Input Requirements

The Steady State Model was tested by comparing the operational policies it produced with those developed by the CSUDP model. This would insure that the basic dynamic programming computations of the model were performing as designed. The model was also tested by simulating the developed policies using the Lake Nasser Simulation Model.

Initially, inflow data used for the evaluation were taken as naturalized monthly streamflows at Aswan for the period January 1892 to December 1982. Five monthly inflow states were selected by the Steady State Model developers and the corresponding transition probabilities then calculated. These *naturalized* inflows are adjusted for annual Sudanese abstractions of 14.8 BCM following the specified monthly distribution pattern. Irrigation demand varies seasonally and is determined according to the cropping pattern and consumptive use. Energy targets, on the other hand, are assumed to be constant over the year.

In order to develop the input data file for the CSUDP model, discrete classes of monthly inflows to Lake Nasser and the corresponding transition probabilities are entered through an interactive menu driven data editor provided with the CSUDP code. Also entered are the control variables such as the type of both the problem and objective function, upper and lower limits on storage and release, the maximum number of iterations for the successive approximations procedure, various splicing and tie breaking options and the desired risk levels. Other data, such as monthly Sudan abstractions, monthly irrigation demands, monthly energy targets and curves for interpolation of power generation, are entered via Subroutine READIN and transferred via common block to the other user supplied subroutines.

The primary inputs to the Lake Nasser Simulation model are the inflow records and the optimal operational policies from the dynamic programming models. Four inflow sequences were used in the simulation: (i) the historical record from 1892 to 1982; (ii) the historical record for the period 1967-1989; (iii) the period 1980 to 1989 to reflect recent drought conditions; and (iv) a synthetic inflow record of two hundred 20 year samples of stochastically generated inflows, based on the historical period 1871 to 1989.

4.2 Steady State and CSUDP Model Results

The Steady State Model was operated for a maximum of 25 cycles, thereby assuring convergence to stationary operating policies. Each cycle requires around 20 minutes on an IBM compatible 386-33 Mhz microcomputer. A sample of the optimal release policies for the months of January [month 1] and September [month 9] are shown in Figures 3 and 4.

The CSUDP model was run using the same data as the Steady State Model. The same number of convergence cycles were used, but with inclusion of the risk analysis feature of the program. The selected risk levels for both α_1, α_2 were 0, 10, and 20%, respectively. It should be noted that these are parameters only and do not reflect the *actual risk*, which can only be evaluated through Monte Carlo analysis and long term simulation. Actual risk levels will tend to be considerably lower than these parameters estimates. Samples of the output for the months of January and September are shown in Figures 5 and 6.

Comparison of these figures shows that the CSUDP policies are considerably *smoother* than the Steady State Model policies. This can be attributed to the splicing option used with CSUDP which allowed policies to be computed within a final discretization interval of 0.01 BCM on releases, whereas the Steady State Model was confined to discretizations of 0.2 BCM. Additionally, 50 state discretization intervals were used within CSUDP, whereas the Steady State Model is limited to 25. This also contributed to the *smoothness* of the CSUDP policies. Examination of these policies also shows that the Steady State Model tends to release higher quantities than needed for irrigation at lower levels in the reservoir than the CSUDP policies. It would be expected that this would result in the CSUDP policies providing higher storage levels in the reservoir.

4.3 Simulation of Optimal Policies

The optimal policies developed by the two dynamic programming models were tested using the Lake Nasser Simulation Model. Minor modifications were incorporated into the simulation model to accommodate changes, including: altering the number of discretized storage levels, setting Sudan abstractions to zero for the case of zero release policies and allowing input of target end-of-month storage policies as an alternative to the optimal release policies produced from the Steady State Model. CSUDP has an additional feature which allows stochastic dynamic programming problems to be run in the *inverted mode*, which generates these kinds of policies.

Simulation was carried out over the 23 years of the historical inflow record from the year 1967 to 1989. Annual and monthly graphs for energy, storage, release and shortages are shown in Figures 7 to 10. From the graphs it can be seen that the CSUDP model (for all risk levels) tends to produce more

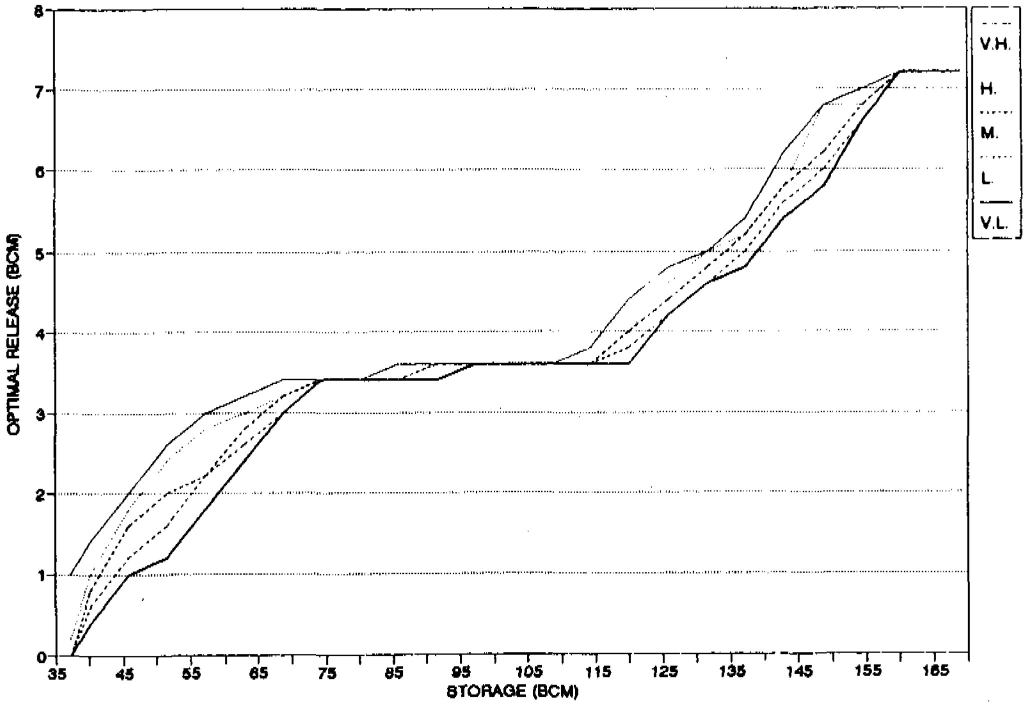

Figure 3. Policies from Steady State model for January conditioned on December inflows

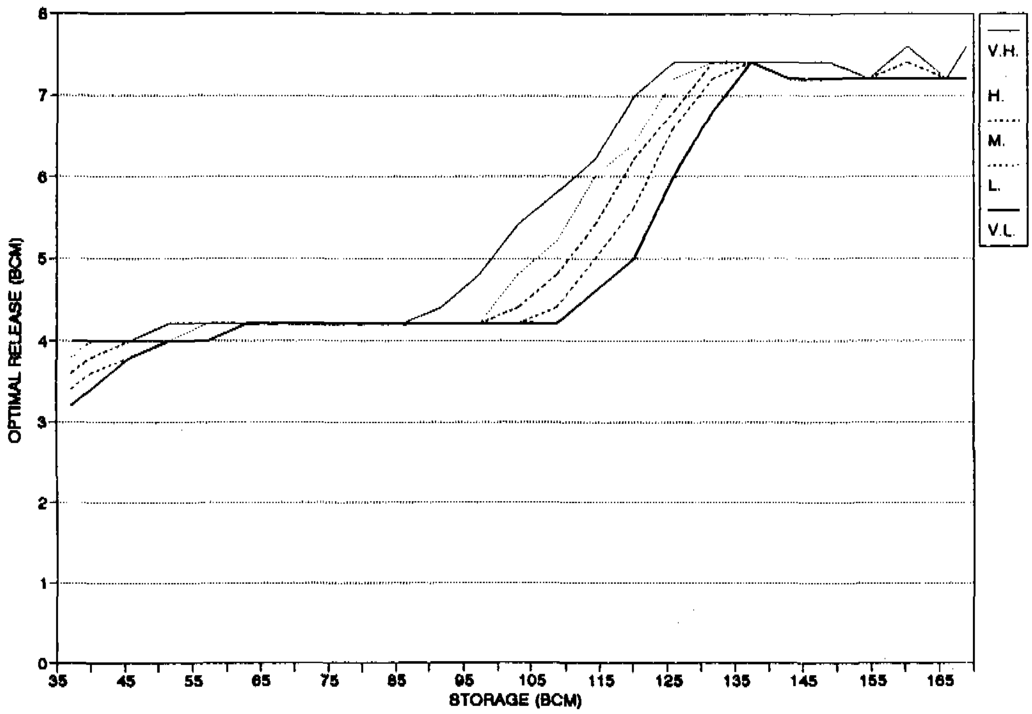

Figure 4. Policies from Steady State model for September conditioned on August inflows

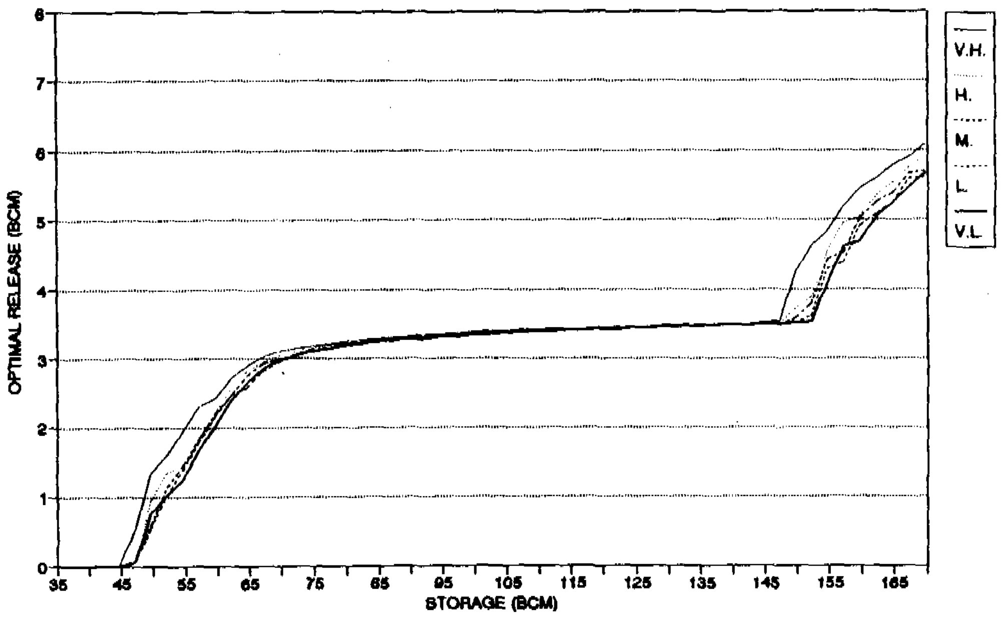

Figure 5. Policies from CSUDP model for January conditioned on December inflows

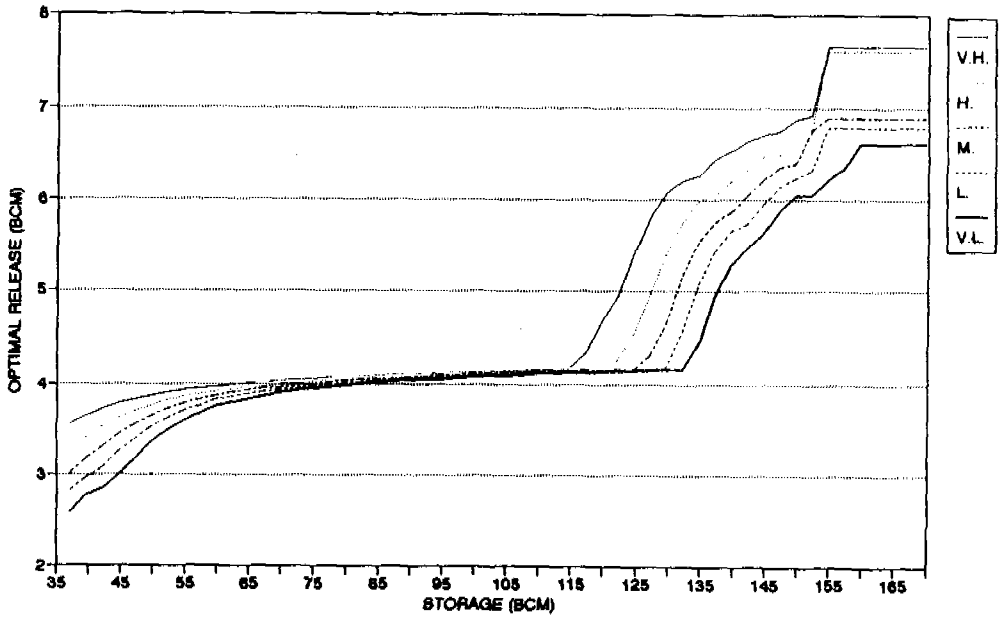

Figure 6. Policies from CSUDP model for September conditioned on August inflows

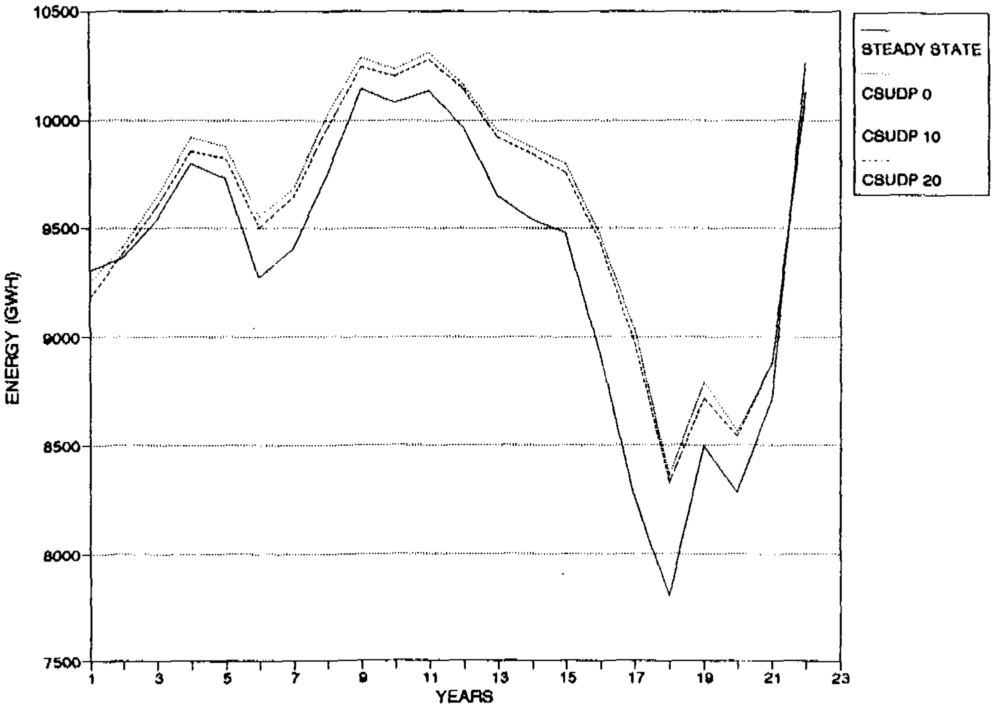

Figure 7. Total annual energy

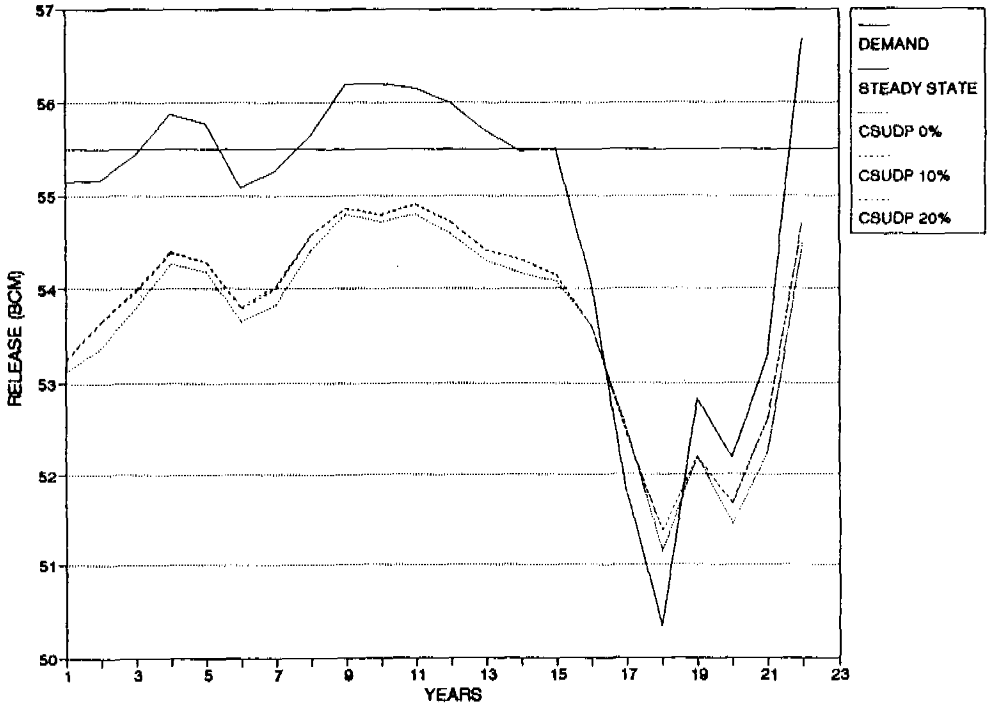

Figure 8. Total annual release

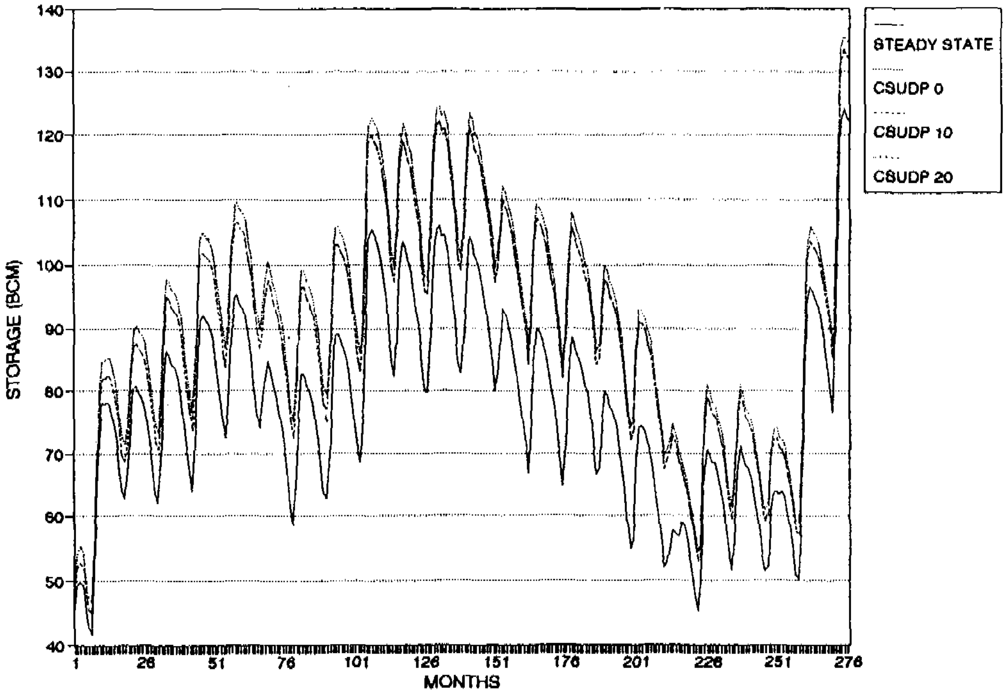

Figure 9. Monthly storage

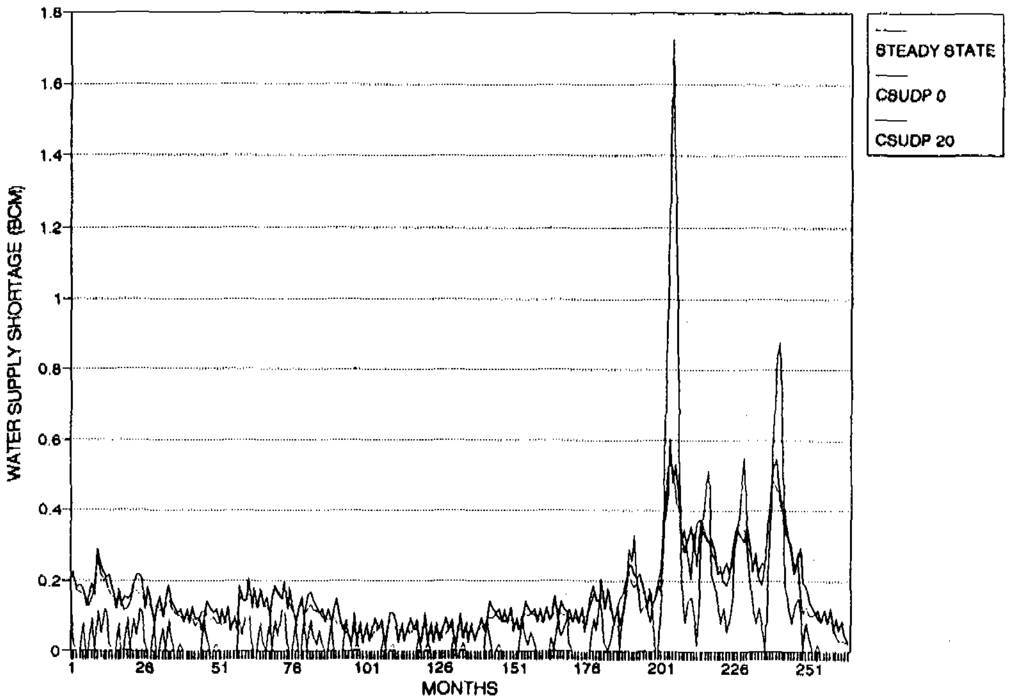

Figure 10. Monthly water supply shortage

energy than the Steady State model. The CSUDP policies also tend to maintain higher levels of storage in the reservoir, but release less. Since this results in more energy production, it appears that the power equations are more head-sensitive than flow-sensitive in the normal operating ranges of the reservoir.

It is apparent from Figure 10 that CSUDP produces so-called *hedging* policies that slightly increase shortages early in the period in order to avoid large shortages at the end when inflows are low. This is consistent with the type of objective function used, which being a squared-error criterion tends to encourage high frequencies of small shortages in order to avoid large shortages. It is interesting that although both models are using the same objective function, the Steady State Model policies do not exhibit a hedging capability.

As a further test, the optimal policies from both models were evaluated using the synthetically generated data set. The results are presented as frequency analyses on annual energy and water supply shortages generated from the Lake Nasser Simulation Model under both CSUDP and Steady State Model policies. Figures 11 and 12 provide the results, which again confirm the higher energy levels produced with CSUDP. Annual water supply shortage plots show that CSUDP policies provide a lower risk of high shortages, but a higher risk of small shortages, as compared with the Steady State Model. Frequency analysis results for each calendar month are not shown, but results show the same tendencies as the annual results.

4.4 Modified Objective Function

Additional tests were carried out to investigate the reasons for differences between policies generated by the Steady State and CSUDP Models. Attempts were made to convert the Steady State Model from an average cost to a total cost criterion. The function $F_n^{\beta}(S_t^0, Q_{t-1})$ in equation 9 was set to zero (which corresponds to the FORTRAN variable *ZETA* in the code). This term represents the dynamic programming optimal return function for a nominal storage and inflow state and is subtracted from the optimal return function. The resulting optimal release policies were run through the simulation model to compare with the basic run. The annual release and energy, water supply shortages and monthly water shortage plots are given Figures 13 and 14 and labeled as *vnall*. Unexpectedly, the results display even further deviation from the CSUDP results. It was concluded at this point that there were fundamental differences between the two models that could not be rectified by simply setting the *ZETA* term to zero.

An additional run with CSUDP was made with the following assumptions: restriction of the discretization interval on release to 0.2 with no splicing, an annual discount rate of 7% on the objective function, and a risk level of 20%. The results are not shown, but they generate increased releases, less energy production and less water shortages when compared with the basic CSUDP run. Consequently, the annual and monthly water shortage patterns were quite close to the basic Steady State run. The use of the coarser discretization interval tends to give a zero slope to the operating rules in the midrange of storage, which greatly reduces the small shortages noted in the basic CSUDP run. Use of the discount rate tends to approach the highly discounted effect of the average annual cost criterion used in the Steady State Model.

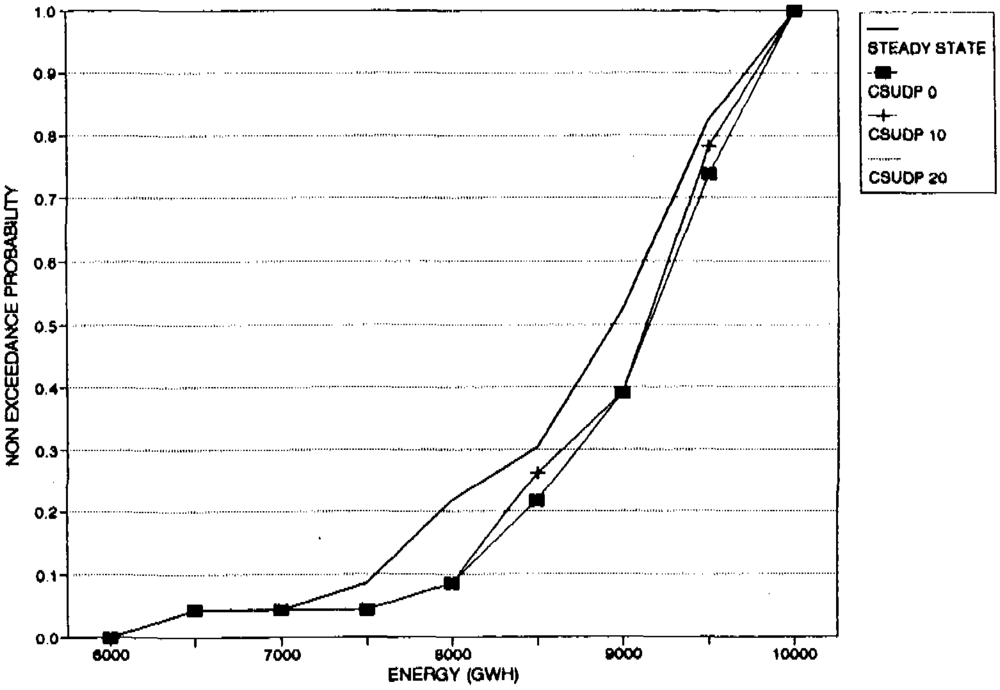

Figure 11. Total annual energy frequency

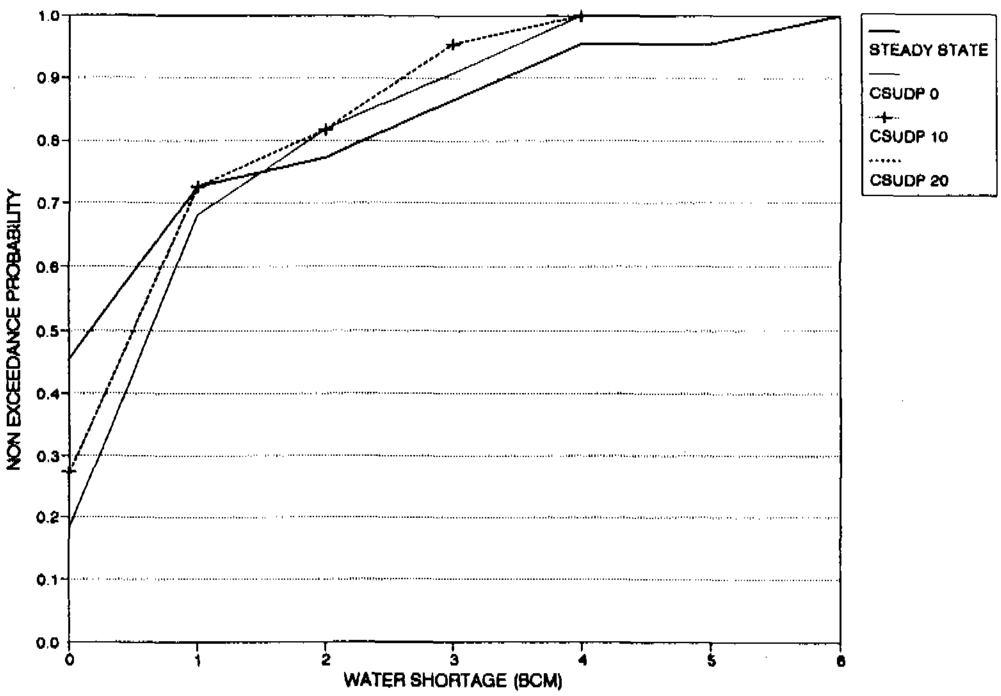

Figure 12. Total annual water shortage frequency

Figure 13. Total annual energy

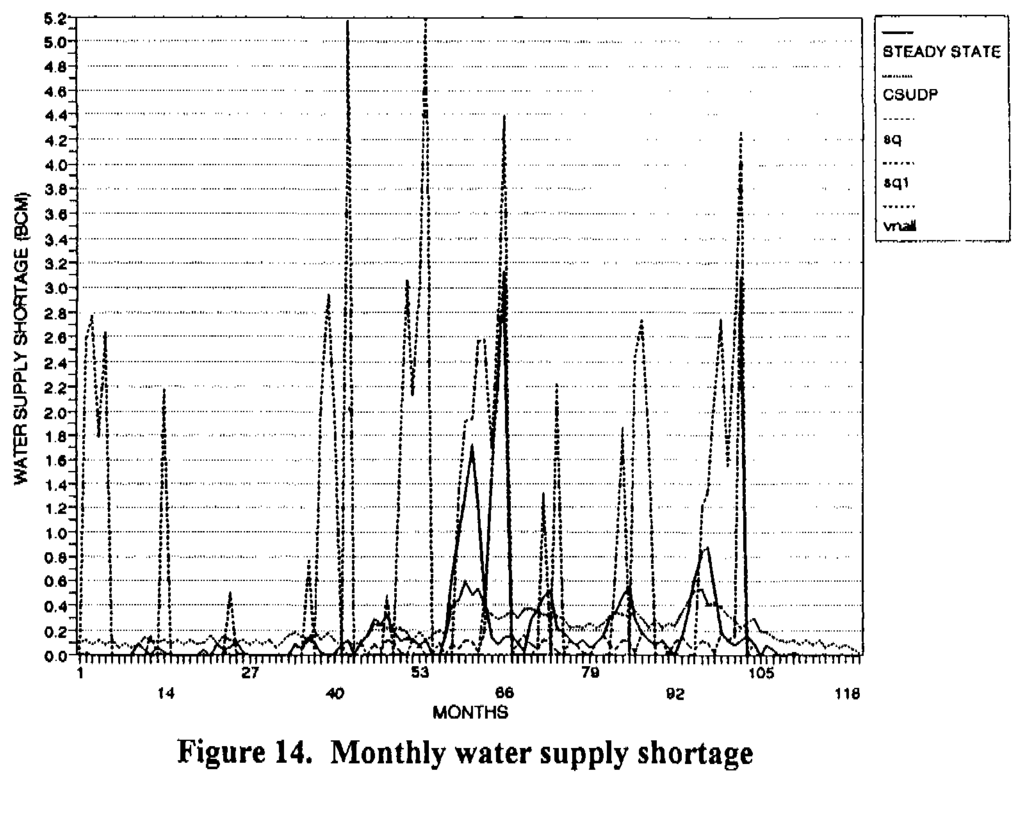

Figure 14. Monthly water supply shortage

It was hypothesized that one of major reasons for the difference between the basic Steady State and CSUDP policies is that the average cost criterion artificially induces a high degree of discounting of future costs by subtracting the *ZETA* term. In effect, this nullifies the *smoothing* effect of the squared penalty terms used in the objective function by selecting decisions at each stage with little regard to future impacts of those decisions. In order to test this hypothesis, it was decided to modify the objective function for the basic CSUDP run by setting penalty coefficients b_1, b_2 to 1 and minimizing the total *absolute* deviations from the ideal energy and irrigation targets. Two risk levels were used in these runs with CSUDP: 0% and 20%. The resulting optimal policies were run through the simulation model for each risk level, respectively. These final runs provided results that corresponding closely with those of the Steady State Model. These results imply that although the objective function used in the Steady State Model is the one which minimizes squared deviations, the way the optimal return function is calculated in this model makes it more like an objective of minimizing absolute deviations. It is concluded from these results that it is the average cost criterion used in the Steady State model that creates discrepancies with results from the CSUDP Model.

5. SUMMARY AND CONCLUSIONS

This study has been directed to evaluating and validating the application of stochastic dynamic programming for developing optimal reservoir operational policies for the High Aswan Dam. The Steady State Model developed for the Egyptian Ministry of Public Works and Water Resources is compared with a generalized dynamic programming model developed at Colorado State University called CSUDP. Both models are applied to the High Aswan Dam using an identical data base and the resulting optimal operating policies evaluated using the Lake Nasser Simulation Model. This model is used to test if optimal policies developed satisfy the Steady State model objectives within the established constraints.

Basic simulation runs indicate many differences between operation policies produced by each model. The most important reason for these differences is the use of an *average cost per period* criterion in the Steady State Model, as opposed to the use of total cost criterion in CSUDP. It has been shown that use of the average cost criterion within stochastic dynamic programming can produce misleading results. The objective function selected as the basis for operational analysis of the High Aswan Dam utilizes an L_2 norm, which performs a *squaring* operation on irrigation and energy shortages. This effectively encourages what are known as *hedging policies,* which attempt to reduce the likelihood of incurring large shortages in irrigation deliveries and energy production by allowing higher frequencies of small shortages. In fact, because of the use of the average cost criterion, the Steady State Model is not capable of producing the type of optimal hedging policies for which it was designed. That is, the Steady State Model does not properly reflect the objective function selected by the Ministry of Public Works and Water Resources as the basis for operation of the High Aswan Dam.

Evaluation of the optimal operating rules produced by both models using the Lake Nasser Simulation Model clearly confirms that CSUDP is producing hedging policies which attempt to minimize the maximum shortages. The Steady State Model produces no shortages during average and wet periods and large shortages during dry periods of below average inflows and low storage levels in Lake Nasser. As a further confirmation, the objective function in the CSUDP Model was modified to reflect an L_1

norm, which simply attempts to minimize total *weighted* shortages over the operational horizon. As expected, the operational policies from this objective function were almost identical to those produced by the Steady State Model, even though the Steady State Model continued to utilize the L_2 norm.

ACKNOWLEDGEMENTS

This work was conducted under an Interagency Personnel Agreement between Colorado State University and the U.S. Bureau of Reclamation. The overall work effort is described in Action Memorandum No. 33 as a joint project between the Egyption Ministry of Public Works, the U.S. Agency for International Development, and the U.S. Bureau of Reclamation. Dr. Bayoumi Attia, Project Director, Planning Studies and Models Project, Ministry of Public Works and Water Resources, provided all necessary models and data for successful completion of this project, as well as valuable guidance and feedback. We are also grateful to U.S. Bureau of Reclamation staff, including Wayne Cheney, Joe Wensman, and Donald Frevert, for their support throughout the duration of this project. Maja Tawfik, Munir Bhatty and Tarek Salem contributed to various aspects of this study, and their help is gratefully acknowledged.

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FAST BASIN MODEL FOR MANAGEMENT OF THE RESERVOIRS ON THE SEINE RIVER AND TRIBUTARIES

Hubert J. Morel-Seytoux,[1]

ABSTRACT

Even in the moderate climate of France severe droughts have been observed in recent years in the Loire and the Seine River Basins. The Interdepartmental Institution for the Barrages and Reservoirs in the Seine River Basin, I.I.B.R.B.S., in short the "Institution", has been concerned with the occurrence of these droughts at a time of increasing demands for water in the Paris urban region. A study was commissioned to investigate how to manage optimally the operations of the reservoirs for flood protection and for low flow augmentation during the dry weather months. The paper reports on the philosophy that governed the specifications of the study and some of the methodologies that were developed for that purpose.

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1. INTRODUCTION

In April 1993, the city of Paris (Agence "Les Grands Lacs de Seine") and the Seine River Basin Agency ("Agence de l' Eau Seine-Normandie") contracted a study to the consulting firm B.C.E.O.M., Montpellier, France and the national research organization C.E.M.A.G.R.E.F., Lyon, France, to investigate the **optimal joint** operations of the reservoirs on the Seine river and its tributaries: Aube, Marne, Yonne (see Figure 1).

Of major ,but not sole, concerns to the two agencies, were the flood protection of the Paris metropolitan area and its water supply, as well as the environmental and ecological aspects of the lake and river waters. Major flooding of the city of Paris, such as occurred in 1910 (see Figure 2), may still happen but rather infrequently. On the other hand a not uncommon situation is the flooding of the roads that follow the banks of the Seine through the very historic part of Paris, as happened this year in January. Imminent anticipated flooding, and consequent safe closing of these roads, creates a major nuisance to the automobile traffic in the city.

In the summer, and specially in the Fall, the natural flows of the Seine are quite low and without augmentation, through releases from the upstream reservoirs, may fall below the drinking water requirements of the metropolitan area roughly once in a hundred year. At present time it is indeed estimated that this demand is about equal to the value of the natural 100 year low flow. Yet it is anticipated that the size of the population of the metropolitan area will continue to increase over the next twenty years. Given that the current total reservoir capacity of the reservoirs is quite limited and that only about 25% of the drainage basin is actually controlled by the reservoirs, one can be concerned both about floods and droughts for the region.

The saving grace is the moderate climate of the basin which for centuries has naturally prevented the region from major catastrophes. However with a growing population and changing land uses, which tend to reduce the extent of the flood plains and increase river flow depletion for irrigation, there is no guarantee that such a fortunate situation will continue into the future. A study was necessary to **describe the natural and managed conditions** in the basin as a preliminary step toward action to alleviate future potential problems. Thus the first study, to terminate in September 1994, was the development of a fairly complete hydrologic model for the basin (i.e. rainfall-runoff, aquifer return flow and river flow propagation) and of the capability of formulation of "optimal" strategies of management under present conditions and future scenarios.

2. REQUIREMENTS

The Institution "Les Grands Lacs de Seine" is responsible for the operations of the reservoirs (diversions into the reservoirs and releases from them) within a set of administrative regulatory rules ("reglements d' eau") and under the advice of a COTECO (Technical advisory coordinating committee ; "Comite technique de coordination"). This committee meets about once a month and possibly more often in case of urgency. It includes representatives from various operational and regulatory branches of government and from a broad section of the water users in general.

At such meetings the Institution presents its plan of operations for say the coming weeks. The various members may make suggestions for deviations from the plan to which they were just exposed. The pros and cons of such deviations, as to their likely impact for the future satisfaction of the numerous objectives that are to be met, are then debated.

One requirement of the model was that it could be run during such meetings so that these impacts can be quantified and demonstrated graphically to the members of the committee. Naturally this is not feasible unless the model can be run rapidly during the meeting itself. As the Institution did not wish to have a computer solely dedicated to that operation it was specified that the model could be run on a standard Personal Computer. It was also specified that the model could be run on a daily basis over a time horizon of 14 months in a matter of a couple of minutes. There lied a major challenge for the development of the model: How to design it so that it could run so quickly without a **significant** loss of accuracy.

3. EXAMPLE OF TECHNIQUES USED TO MEET THESE OBJECTIVES

It is not possible in this modest space to discuss in details all the aspects that went into the components of this comprehensive model. Rather we shall briefly outline just a particular one, for the conception of which the author was largely responsible.

The component is that for the propagation of flows in the rivers. The rivers are treated as behaving in a linear fashion; however the system is not assumed to be time-invariant. Each river is treated as a succession of reaches, mathematically represented as Linear Reservoirs (Morel-Seytoux, 1993a) but whose time "constant" is varying with time (Morel-Seytoux, 1993b) because it is related to the discharge occurring in that particular reach at that time. This formulation leads to the simple formulation (of the auto regressive form) :

$$\overline{O}(n) = c_O\overline{O}(n-1) + c^v\overline{I}(n) + c^o\overline{I}(n-1) \tag{1}$$

where $\overline{O}$ stands for mean daily outflow rate from the reach and $\overline{I}$ for mean daily inflow rate into the reach, during day n or n-1. The coefficients are defined as :

$$\rho_n = \frac{C_{n-1}}{C_n}\left(\frac{1+\Delta_n}{\Delta_n}\right) \tag{2}$$

with $\Delta_n = C_n - C_{n-1}$ (3) and defining further :

$$u_n = C_{n-1} - \rho_n C_n \quad \text{and} \quad v_n = \frac{u_n}{u_{n-1}}$$

finally : $c_O = \rho_{n-1}v_n$, $c^v = \dfrac{1-u_n}{1+\Delta_n}$, $c^o = \dfrac{u_n - \rho_{n-1}v_n}{1+\Delta_{n-1}}$

The time constant of the linear reservoir representing each reach river is related to hydraulic parameters and to discharge by the relation :

$C = \frac{3 n_M^{\frac{3}{5}} W^{\frac{2}{5}} L}{5 S^{\frac{3}{10}}} Q^{-\frac{2}{5}}$ (4) where n_M is Manning' s n, W is the river width, L is the river length , Q is the discharge and S is the river slope.

Naturally this is the basic form that must be modified to account for lateral expansion into the flood plains, aquifer return flows from the alluvial aquifer, tributary inflow from the riparian watersheds, etc. The model also includes the description of the runoff generation in the upstream parts of the basin, soil moisture accounting, aquifer recharge, etc.

4. CONCLUSIONS

The model is not yet fully operational as project completion deadline is mid-September 1994. The hydrologic model was calibrated apparently successfully and runs indeed fast as required at least during early tests made in the study. However the optimization component technique is time-consuming. In a later phase a different optimization technique will be tried which incorporates the particular structure of the hydrologic model in the very core of the optimization procedure. That optimization technique is a new form of the "Constrained Calculus of Variations" technique combined with the "Marginal Value Approach" (Morel-Seytoux, 1994).

ACKNOWLEDGMENTS

This report is too short to render justice to this comprehensive project. It is only meant to be a brief introduction to this piece of work. I want to thank Mr. Jean-Louis Rizzoli, Adjoint à l'Ingénieur en Chef, Service des Barrages-Réservoirs, Les Grands Lacs de Seine, for having provided me with the opportunity to participate in this challenging project as a technical adviser. I want to think also Mr. Villon, Engineer with the Institution for many good discussions and some very interesting site visits to the reservoirs and the rivers. At the BCEOM I was privileged to work closely with Mr. Guy Chevereau, overall project leader, and Mrs. Mireille Raymond and at CEMAGREF with Mr. Jean-Baptiste Faure, responsible for the definition of the optimal release policies. Those interested in this modeling approach should contact these various individuals for more information about the many other aspects of the project and obtain more details about it. The author will be glad to provide mailing adresses for these individuals and their respective companies.

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DU BASSIN DE LA SEINE

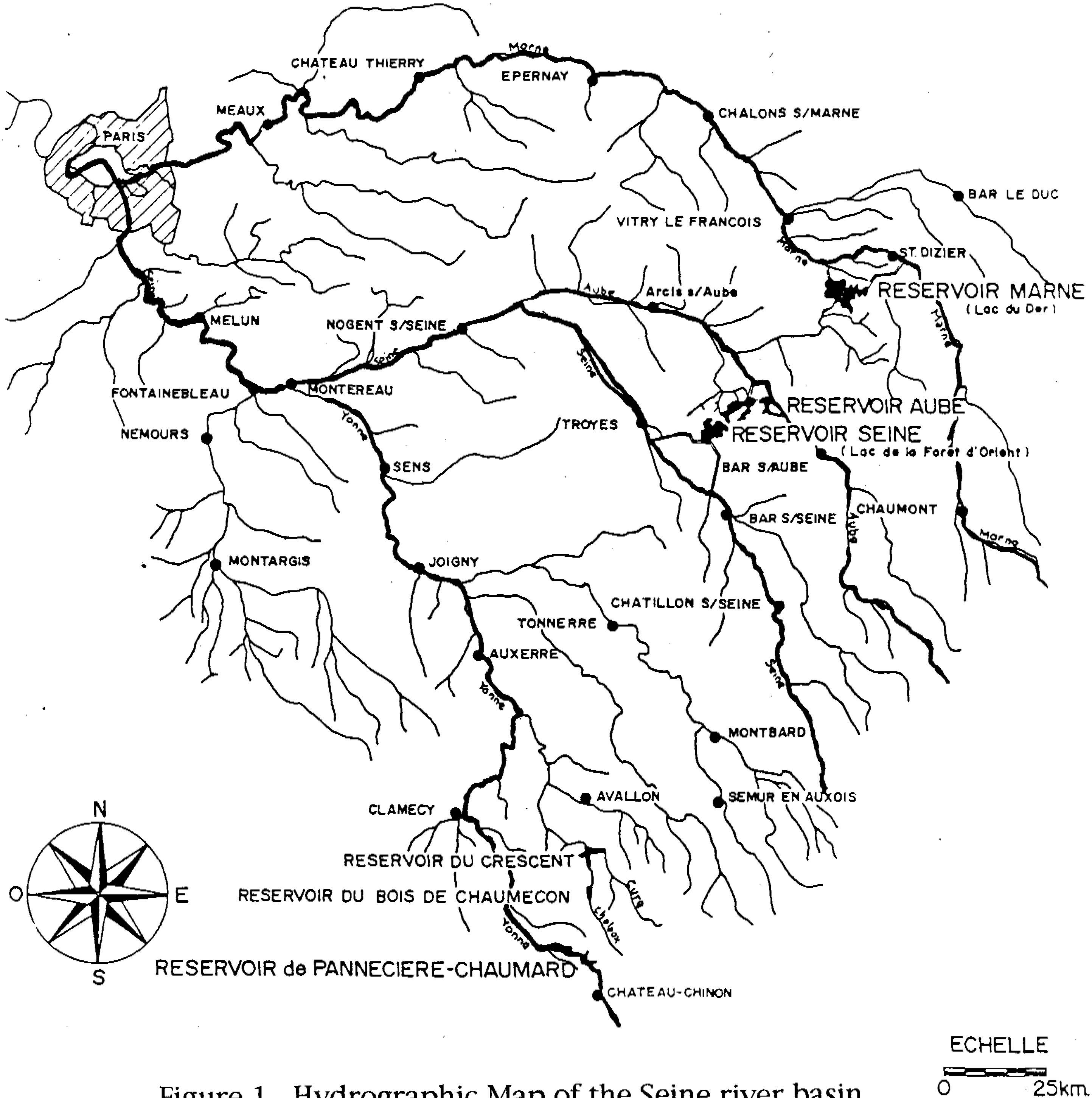

Figure 1. Hydrographic Map of the Seine river basin showing the Seine, tributaries, cities and location of reservoirs in the system.

:ARRAGE - RESERVOIR

AUBE

Figure 2. 1910 Flooding scene near Saint Dominique street in Paris.

PROGRESSIVE MARGINAL WATER SOURCES DEVELOPMENT IN ARID ZONES

Gideon Oron[1], Abraham Mehrez[2], Jack Brimberg[3]

ABSTRACT

A management model is presented for the optimal development of marginal water sources in arid zones in conjunction with minimizing the dependence on high quality water. The marginal sources include saline ground water, treated wastewater and runoff water and are required to augment a limited supply from regional high quality local sources. The objective is to minimize operational and capital costs while simultaneously allocating a conventional regional supply in a best way among a set of local sites. A novel aspect is the consideration of water quality as an additional constraint in the decision model. In this way an optimal investment strategy for marginal water sources development and use is obtained while satisfying quality requirements at the individual sites.

The model formulated takes the form of a mixed binary integer linear problem. The main purpose of the presented model is to delineate a methodology for marginal water considerations and development in arid zones. Water qualities, supply and demand for diverse uses and related costs are of primary importance. Several simplifying assumptions are made, such as aggregation on an annual basis, in order to cope with the essential features of the problem at a reasonable level of complexity. These assumptions may be relaxed at a later, more-detailed stage of analysis.

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1. INTRODUCTION

1.1 Marginal Water in Arid Zones

Fast multiplying demand for high quality water, linked with natural shortage and quality deterioration in arid zones, has resulted in excessive usage of conventional sources. The demand can primarily be satisfied when marginal sources such as saline, waste and runoff waters, are put to use and regional considerations are taken into account. Regional water planning, development and management has therefore become an important uncertainty in arid zones with limited high quality sources. Development of the various sources must be implemented gradually by taking sequential steps, subject to factors such as stability of supply, quality of water, future development, agricultural cultivation potential, and related economic aspects (Coe, 1990; Oron et al., 1991).

The gap of water shortage in arid zones can be concluded by implementing two major directions: (a)conducting water from external sources, using large conveying systems; (b)gradual development of local marginal water sources, subject to regional needs and future prospects. The main water supply to the Negev Desert region in southern part of Israel is maintained from the National Water Carrier (NWC). Water is pumped from the Sea of Galilee and merged with the coastal and mountain aquifers waters in the northern and central areas of the country. The motivation for integrating water supply through the NWC was to satisfy the needs of the whole country for the foreseeable future. However, several drought events, akin with increasing demand in all sectors of the economy has resulted in over-pumpage of the water supply system in recent years. This is exemplified by a decrease in the water tables of the coastal and mountain aquifers, as well as a decline in the levels of the Sea of Galilee and the Dead Sea downstream. Nativ and Issar [1988] concluded that current system is not capable of expanding much further to supply the expected water needs toward the end of the century, not to speak beyond the year 2000.

In order to reduce the dependence on the NWC and alleviate the problems associated with over-pumpage, it has become necessary to develop the marginal water sources existing in the Negev. These sources have the following characteristics:

i) *Treated wastewater* (TWW). Treated wastewater can be reused for a large pattern of possibilities, primarily for agricultural irrigation (Asano et al., 1992). The major drawbacks of TWW are the high capital investment in the treatment facilities and equipment, the dual piping system required to distribute it separately from potable water, effluent quality control and requirements. The nutrients contained in the wastewater are, however, beneficial for agricultural use (Crook, 1985; Oron et al., 1991).

ii) *Runoff water* (ROW). Runoff water is obtained during sparse rainfalls events in the wet (winter) season. Relatively high capital investments are required for the collection, storage and distribution of the water to the consumption sites. The stochastic nature of runoff water supply raises additional difficulties to use this water source efficiently. The high-quality of the water, and the potential to retain large volumes is advantageous in regions with scarce conventional waters.

iii) *Saline ground water* (SGW). Saline water can be found in deep aquifers (~ 1000 m) throughout the Negev and Sinai Deserts (Issar and Adar, 1992). The high variable costs for pumpage and the lower quality of the water generates extra difficulties. The main advantage stems-from the huge volume stored in the Negev (billions of cubic meters) and the possibility of utilizing the water for further desalination or for irrigation of a broad pattern of crops. Advanced application techniques are frequently implemented to use the SGW for irrigation (Pasternak et. al., 1986).

1.2 The Purpose of The Work

In this work a management model was developed for optimal water sources development in arid zones. The aim is to use gradual phases in development and implementation of the non-conventional sources. The implementation is simultaneously subject to a series of technological and environmental constraints and financial alternatives.

The objective of the model is to determine the optimal distribution of a limited high quality water supply from the NWC among the local sites in a dry region while simultaneously planning the optimal development of supplementary marginal sources at these sites. A case study using six representative sites from the Negev region is analyzed in detail. Based on a sensitivity analysis of key parameters in the model, some important conclusions are drawn.

1.3 Models for Optimal Water Sources Utilization

Optimization of water sources and distribution systems attracted the attention of managers and scientists recently. Various algorithms for branched or closed loop systems have been tested to arrive at optimal layout and dimensions of the hydraulic components along with minimal operation expenses (Carpentier et al., 1985; Hiesel and Plate, 1990). Lansey and Mays (1989) addressed the complexity of the problem and reviewed several solutions. They adopted concepts of optimal control theory and implemented a generalized reduced gradient method for the overall optimization of the network.

Studies concerning optimal utilization of limited water resources and networks include implementation of the graph theory, decomposition of complex problems, out-of Kilter algorithm (OKM), dynamic programming (DP) and others (Kessler, 1988; Carpentier and Cohen, 1984; Labadie et al., 1986; Rao et al., 1990; and Minoux, 1989, respectively). DP was implemented to solve a multicrop and multistage agricultural scheme. The problem was decomposed into two competitive water levels (seasonal and inter seasonal), and solved for arid conditions. Multi-objective optimization (MOP) is frequently applied to solve complicated problems in the area of water resources associated with water allocation and transportation. In the mixed simulation-optimization model by Hiessl and Plate (1990) MOP was the second phase used to determine optimal crop pattern under a set of constraints.

Management expansion models are adopted for anticipating the possibilities for new developed sites (Kim and Clark, 1988). Optimal capacity expansion of water supply system, based on hierarchical phases was suggested by Martin (1987) and by Dyer et al. (1990) for gas and oil systems. The model by Martin (1987) is based on a dynamic programming algorithm which computes the least costly capital investment plan under optimal operating expenses.

Various approaches have in the past been undertaken to plan, evaluate and control regional water systems. However, studies concerning regional systems with various water qualities are limited (e. g. Schwarz et al., 1985; Vieira and Lijklema, 1989). The objective herein is to present a mathematical model described in a mixed binary integer linear form, for use as a tool in decision-making and in planning the development of marginal water sources at a regional level for a typical arid zone. A unique feature in the model is the addition of constraints on net water quality at each demand site, to account for the different qualities of the alternate sources. In addition, the special structure allows the problem to be decomposed into a set of simple local problems. It provides insight into the relation between the local and regional levels of decision making. The decomposition characteristics may also be exploited in an efficient solution algorithm for larger scale similar problems.

2. WATER SOURCES DEVELOPMENT MODES

Three levels of planning and development of marginal water sources in arid zones can be insinuated. At the most detailed level, the individual demand site is established to provide for development of marginal local sources for domestic and agricultural consumption. It is assumed that the supply from the NWC cannot meet current demand here, or projected demand in short to intermediate range (up to 5 years), so the marginal sources must be developed soon under the site's own initiative. Alternatively, the price charged for the water from the NWC may have risen, because of increased demand on this system, to the point where alternate sources may be economically feasible shortly or in the near future. Because of the short time horizon a static model is proposed at this level of planning.

At the next level of aggregation, planning at a regional scale is considered. The total supply to the region from the NWC is limited, and the problem therefor becomes one of allocating this supply to the individual demand sites in an optimal manner. The current demands (or short-to-medium range projections) are used, so that the problem here is also of a static nature. The two-levels of planning are inter-related, since the regional decision to make available an explicit volume from the NWC to a given site will affect the decision on which marginal sources to develop locally. Thus, an integrated decision model combining the two-levels is required.

The highest level of aggregated planning involves the long-range, strategic development of the marginal water sources in the Negev Desert. The problem formulation here is of a dynamic nature, since the time element must be considered (Schwarz et al., 1985). Input data into the model are the forecasted demands for water throughout the region for several years. The aim at this level would be to investigate the feasibility, optimal timing and location of large, new water facilities. Examples of such facilities include regional wells and pumping stations, and large desalination plants. Long-range strategic planning of marginal water sources is beyond the scope of this paper. The new sources should be developed subject to the national water needs and principles of already existing water supply system (Figure 1).

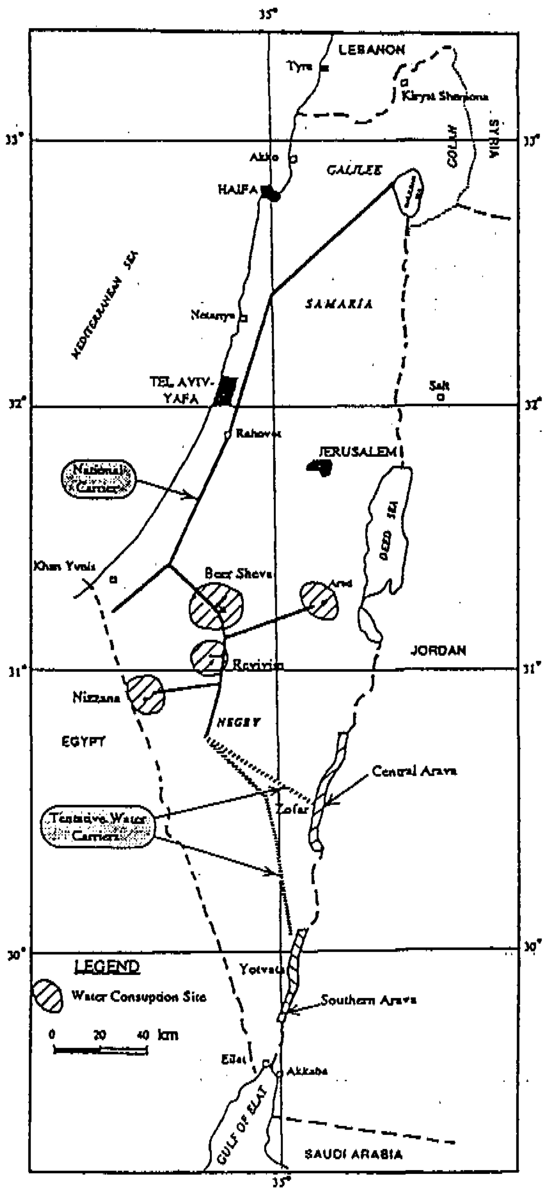

Figure 1. Consumption sites in the arid Negev Desert, and water supply from an external source (the NWC), Israel

3. THE MODEL FOR REGIONAL WATER PLANNING

3.1 General Structure of the Model

As noted above, the local and regional plans ought to be integrated in order to obtain an optimal distribution of the limited water supply from external sources (the NWC), while simultaneously choosing the best local marginal sources to develop in order to make up the deficit. Actually, the purpose is to determine, with minimal expenses, the two main decision variables. In the model, these are denoted by q_{ij}, representing actual supply (m^3/year) to demand site i from source j, and by y_{ij}, which is a binary integer variable to flag new capacity at site i from marginal source j ($y_{ij} = 1$, if new capacity is added, $y_{ij} = 0$ otherwise). A mixed integer linear program is proposed to meet this requirement. Although most akin processes are nonlinear, it is anticipated that a mixed integer linear model will yield reasonable results and extra directions for further water resources development. Some simplifying assumptions which entitle a concise mathematical formulation of the problem are discussed.

3.2 Assumptions of the Model

The major purpose of the paper is to formulate a model which adequately describes the essential features of the problem, and provides a fundamental tool for decision-makers in planning the development of marginal sources under various limits of supply from external sources (the NWC). Thus, it was considered adequate at this stage to aggregate the level of detail in the model on an annual basis. In addition, the problem was linearized by neglecting higher-order effects, such as the quality of water on crop yields (and subsequently agricultural productivity and related profits). This approach is reasonable when the acceptable range of water quality applied (net quality when blending is considered) at the sites is sufficiently tight. For practical implementation at a later stage, a more detailed approach should be considered, with aggregation on a seasonal basis and the estimated effects of water quality on crop yield included. The model was examined subject to several assumptions:

a) All demand sites are connected to the external water sources (the NWC in the case of Israel), or can be connected with negligible fixed costs. This condition applies to the case study presented later.

b) The new capacity v_{ij} which can be added at site i from marginal source j is known and fixed, having been selected separately from a small number of design alternatives. This capacity will be limited by pumping regulations in the case of SGW, population size at the demand site for TWW, and the probability distribution of rainfall for ROW. In addition, budgetary constraints at the site will limit the capacity which can be added. The model is readily generalized to include different capacity alternatives, however, it is not an important feature in the decision process at this stage.

c) Water quality requirement will vary among the distinct crops planted at a demand site, as well as during the life cycle of each crop. It is assumed that sufficient storage and handling facilities exist to meet these individual requirements, and that the costs associated with mixing water from different sources can be neglected. For purposes of analysis, it is also assumed that the quality requirements at a site can be summarized by a net annual quality constraint. It is impartial when there is only a non-essential variability in quality between the seasons.

The regional multi-quality water supply system will be described by a objective (cost) function to be minimized. The cost will be expressed in terms of an equivalent annual amount (EAC_i). Capital investments at the sites are discounted assuming perpetual life for the new assets. Thus, the equivalent annual cost at site i is given by:

$$EAC_i = \sum_j c_{ij} q_{ij} + \tau \sum_{i,j \neq 1} b_{ij} y_{ij}, \quad \forall i \tag{1}$$

where c_{ij} is the variable cost coefficient ($\$/m^3$) for satisfying the demand at site i from source j, q_{ij} is the unknown supply (m^3/year) to demand site i from source j, τ is the applicable rate of interest for discounting purposes, b_{ij} is the capital investment (\$) at demand site i for new capacity from marginal source j and y_{ij} is an directive variable ($y_{ij} = 1$, if a new capacity is added, and $y_{ij} = 0$ otherwise). Attaining the minimal cost is subject to the assumption that the crops selected and planted acreage have been determined, so that each site generates a fixed revenue.

3.3 Model Formulation

The optimization model takes the form of a mixed binary integer linear program:

$$\text{Minimize} \quad Z = \sum_{i,j} c_{ij} q_{ij} + \tau \sum_{i,j \neq 1} b_{ij} y_{ij}, \tag{2}$$

The objective function to be minimized is subject to a series of constraints::

(i) Regional constraint (maximal allowed supply from an external source, the NWC):

$$\sum_i q_{ij} \leq S \tag{3}$$

(ii) Local constraints:

$$\sum_j q_{ij} \leq d_i, \quad \forall i \qquad \text{(demand)} \tag{4}$$

$$\sum_j \rho_{ij} q_{ij} \leq \gamma_i d_i, \quad \forall i \qquad \text{(quality)} \tag{5}$$

$$q_{ij} - M y_{ij} \leq s_{ij}, \quad \forall i, j \neq 1 \qquad \text{(fixed cost)} \tag{6}$$

$$q_{ij} \leq s_{ij} + v_{ij}, \quad \forall i, j \neq 1 \qquad \text{(capacity)} \tag{7}$$

(iii) Non-negativity and integrally constraints:

$$q_{ij} \geq 0, \; y_{ij} = 0,1, \quad \forall i, j \tag{8}$$

where $i = 1, \ldots, N_d$, and N_d is the number of demand sites in the system (i≠j). Here S denotes the maximum total volume (m^3/year) supplied to the region by the NWC, is the total annual demand (m^3/year) at site i, γ_i is the mean quality [measured in units of electrical conductivity (EC), dS/m] required at site i, ρ_{ij} is the quality (EC) of the water at demand site i supplied from source j, M is a very large number, s_{ij} is the existing capacity (m^3/year) at site i from marginal source j (j=2,3,4), v_{ij} is the new capacity (m^3/year) added at site i from marginal source j (j=2,3,4).

Subject to the model structure, the fixed cost constraint ensures a capital investment b_{ij} is incurred in the objective function whenever the supply at site i from marginal source j exceeds its current capacity. The value assigned to M must be sufficiently large so that,

$$M \geq \max_{i,j} \{v_{ij}\}, \quad \forall i, j \neq 1 \tag{9}$$

The capacity constraints at site i ensure that the total annual capacities of the marginal sources here are not exceeded. However, the on-site storage and handling facilities may be adequate if seasonal use of these capacities is uneven, resulting in an infeasible solution. This typically is not a problem and in any case, can be remedied by replacing annual constraints by seasonal ones.

4. A SAMPLE OF RESULTS

4.1 The Analyzed System Layout

Six representative demand sites were chosen from the Negev Desert region to form a case study (Figure 1). The demand sites are listed in Table 1 along with their demands and net water quality requirements. Water demand of each site depends simultaneously on environmental conditions (e.g. mean, maximal and minimal temperatures and evaporation rate) and on soil properties allowing economical cultivation of a variety of cash crops. The capital investment and related operation and maintenance expenses of each of the individual water sources in every specific area depend primarily on the local hydrological conditions, namely depth of ground water and soil structure enabling to withdraw the water by appropriate drilling. The water from the generating points is generally delivered by pumps and pipe networks to the consumption sites. All the water used for agriculture is delivered via closed conduit systems, namely sprinkler and trickle irrigation.

Table 1. Water quality and capacity characteristics for the demand sites

Index i	Site Location	Demand, d_i [x $10^3 m^3/yr$]	Quality Required [γ_i - (EC), dS/m]
1	Central Arava	18,750	2.16
2	Southern Arava	30,800	2.43
3	Nitzana	51,000	2.24
4	Beer Sheva	25,200	3.09
5	Revivim	15,000	1.73
6	Arad	40,000	2.50

4.2 Extreme Situations of the System

Two extreme cases are investigated. These provide some insights into the model properties. First the minimum annual supply from the NWC which allows a feasible solution to the problem is determined. Next the case of an unlimited supply from the NWC is considered. It is reasonably assumed throughout that the demand at each site cannot be completely satisfied by the marginal sources, namely:

$$d_i - \sum_{j \neq 1} \left(s_{ij} + v_{ij} \right) > 0, \quad \forall i \tag{10}$$

4.3 Minimum Annual Supply from the National Sources

One of the purposes of using marginal water sources is to eliminate the dependence on high quality supply. This means that under arid conditions such as in the Negev Desert a larger portion of high quality water from the NWC will be delivered for sensitive purposes such as domestic and sophisticated industrial consumption. For the given levels of demand and quality requirements at the individual sites it is therefore viable to obtain a minimum acceptable annual supply from the NWC. Before proceeding with the calculations, it can be noted that the quality of water from all sources except SGW is superior to the net quality required (γ_i). Thus, if a feasible solution exists to the model, another one can always be found so that the runoff and TWW sources at each site are operating at full capacity.

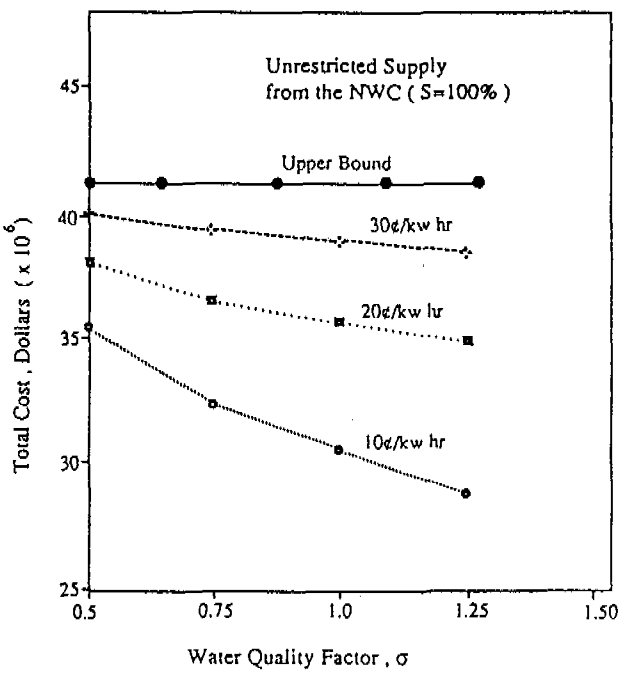

Figure 2. Dependence of total cost on water quality factor σ

As predicted, total cost (Figure 2) decreases as the net quality requirement is relaxed. Furthermore, the slope of the cost curve is steeper at lower energy prices, since economical sources of saline ground water cannot be exploited as quality levels become more stringent. In Figure 3, a general tendency for the volume of SGW to decrease is observed as the net quality requirement is tightened (or σ decreases). Rather surprisingly however, the amount of saline ground water increases as the quality factor σ decreases for 70% of the NWC availability scenario and for a quality (σ) range of 0.75 to 1.0. This counter-intuitive result may be explained as follows: when the supply from the NWC (external source) is limited, a deficit exists which must be made up principally from SGW sources. As the quality requirement is tightened, the capacity of SGW at some sites will be under-utilized, making the corresponding investments uneconomical. The deficit is made up at other sites, where capacities may be used in a more economical manner. Comprehensive results of a case study for the Negev Desert are presented above. Further detailed breakdown of marginal sources at the individual sites, can be found in Brimberg and Oron (1991).

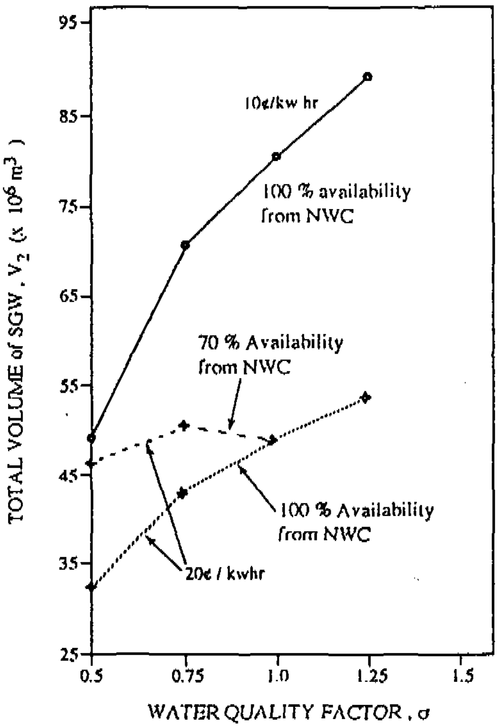

Figure 3. Effect of water quality factor σ on the volume of saline ground water consumed

5. SUMMARY AND CONCLUSIONS

This paper presents a conceptual model attempting to estimate the optimal gradual stages required for marginal water sources development in arid zones in the presence of a limited supply from a conventional regional high quality source. The model here within reflects a potential methodology to deal with water quality and scarcity problems in arid zones. The simplicity of the mixed integer linear presentation of the problem allows to implement it for a broad pattern of cases providing basic water quality, amounts and production costs information is available. Estimates are based on interchange between annual quantities and qualities of waters at the demand sites and subject to annual costs. Certain conclusions can be drawn from the results, which are rather consistent.

1. The most important outcome obtained from the case study is the feasibility and economic viability of saline ground water over a wide range of energy prices and required quality levels. Based on the findings of the case study, it can be concluded that utilization of SGW in a dry region such as the Negev Desert should be increased enormously, through immediate investments in new capacities.

2. Based on current prices, consuming water from the NWC or TWW is also an economical option. Even without restrictions on supply from the NWC, development of a large treatment plants, operated at relatively large capacities adjacent to urban areas (e.g., an approximate capacity of 6.6×10^6 m^3/year close to the city of Beer-Sheva, Israel) is recommended.

3. The given quality requirements and levels of demand can be satisfied with a significant reduction in supply from external sources (the NWC). A feasible solution exists with this supply cut back to about 47% of total demand. However, increased use of marginal sources must be balanced by the increased costs incurred.

4. The capital investments for new capacity amortized on an annual basis contribute only a small fraction of the total equivalent annual cost. The major contribution is derived from the variable costs opponents.

The model presented in this paper includes some simplifying assumptions which might confine its applicability. In the case study analyzed, ROW has been ignored as additional potential source, due to the small capacities attributed to it. Extra efforts are recommended in studying the potential use of ROW and associated costs. Depending on the price of energy, this source may prove to be a preferable supplementary option to saline ground water. A definite amount of judgment is required in selecting some parameters in the model, such as the quality coefficients (ρ_{ij}, γ_i), which are sensitive to a number of factors. Regional development and arid zones settling is primarily subject to decisions made at national levels and under planetary considerations. Under these circumstances water resources development is turned into a prominent integrated component of the whole perception. For strategic purposes, mixed integer linear models can provide extensive information, tendencies of processes and phenomena which naturally are nonlinear. That is mainly true when scarce data is available to come-up with reasonable relationships between the variables of concern. Incorporating forecasting procedures based on limited information in order to substitute the deterministic variables might deteriorate the reliability of the derived results. On top of it general models should be investigated, for example, ones which include the effects of water quality on crop production and profits. Rigorous specification of parameters and additional detailed sensitivity analysis over applicable ranges of values are essential, before final decisions are implemented. For further verification large scale models should be formulated, including all characteristics of the demand sites in the region.

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GEOGRAPHIC INFORMATION SYSTEMS AND EXPERT SYSTEMS HYBRID FOR DEVELOPING A PHYSICALLY-BASED DISTRIBUTED HYDROLOGIC MODEL

Gamal Ibrahim Allam[1]

ABSTRACT

A single event physically-based, distributed hydrologic model has been developed to determine watershed response to a rainfall event. The spatial variability of the physical hydrologic, hydraulic and soil parameters is represented by the Geographic Information System (GIS) raster, grid cells, data structure. Combination of the physically-based equations and GIS analysis capabilities allows the physical and numerical integration of dynamical hydrologic processes throughout the watershed. The model deals with GIS both as programming environment and as data-base. In addition, an expert system shell (ESS) is developed to automate and manipulate the execution of each step of the computations and to ease the task of the unexperienced hydrologic and/or GIS modeler. Results indicate that GIS technology is a very useful technique for developing the distributed hydrologic models besides its capabilities for large spatial data-base management. For future studies, this technique needs some enhancements to its analysis capabilities to accommodate the iteration technique which is required for routing and calibration processes.

INTRODUCTION

Many researches had been done to develop a physically-based distributed hydrologic models using different techniques, finite difference and finite elements in two dimensions and three dimensions. The difficulties of developing these distributed physically-based models are more or less the requirements of a huge data-base for the spatial and temporal variabilities of the hydrologic, hydraulic and soil parameters required for models with this nature.

Applications of physically-based models have been primarily concerned with exploring the implications of the assumptions with an objective of improving the understanding of a particular theoretical construct (Freeze, 1972, 1980; Smith and Hebbert, 1979). The physics on which the equations are based is the small scale physics of homogeneous systems. In real applications of physically-based hydrological models lumping of the small scale physics to the model grid scale will be necessary. There is no theoretical framework for carrying out this lumping of subgrid process for spatially heterogeneous grid squares. After producing GIS

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technique to the research field much of these implications should disappear because of its large capability as spatial data-base. Most of the well known hydrologic models are now under processing to be linked with GIS to improve their data storage and display capabilities. Many researches these days are trying to utilize the GIS analysis capabilities for developing a hydrologic models using these capabilities. Yet simple and mainly empirical equations are used to evaluate the hydrologic response of watersheds (Drayton, Wilde and Harris, 1992).

In this research the physical hydrologic process is applied on the spatially heterogeneous state of each cell to accommodate the definition of physically-based distributed hydrologic model. The accuracy of determining the watershed hydrologic response is a function of the accuracy of hydrologic, hydraulic and soil parameters for each subgrid.

METHOLOGY

To accommodate the complexity and the spatial variability of the physical hydrologic process, this study will divide the watershed into a number of subregions according to the soil and hydraulic parameters. Also, the model is dividing the watershed into its subcatchments for its calculations.

The excess rainfall is computed as the gross rainfall minus the infiltration losses which is the dominant losses during the storm. This model uses Philip's infiltration expression. In this expression infiltration capacity is expressed in terms of sorpitivity and gravitational infiltration as:

$$f_i^*(t) = \frac{1}{2} S_i t^{-\frac{1}{2}} + A_i \tag{1}$$

For saturated soil, S_i and A_i are expressed as:

$$S_i = 2(1-s_o) \left[\frac{5 \; n \; K(1) \; \psi(1) \; \Phi_i(d,s_o)}{3 \; m \; \pi}\right] \tag{2}$$

where the dimensionless sorption sorpitivity $\Phi_i(d,s_o)$ is expressed as a function of soil properties by Eagleson (1978c) as:

$$\Phi_i(d,s_o) = (1 - s_o)^{-5/3} \int_{s_o}^{1} s^d (s - s_o)^{2/3} \, ds \tag{3}$$

which becomes, for integer d only, a function of soil properties as:

$$\Phi_i(d,s_o) = (1-s_o)^d \left[\frac{1}{d+5/3} + \sum_{n=1}^{d} \frac{1}{d+(5/3-n)} \binom{d}{n} \left(\frac{s_o}{1-s_o}\right)^n\right] \qquad (4)$$

for non-integer d, use expression 3 or interpolate. A_i is expressed by Eagleson (1978c) as a function of soil properties as

$$A_i = \frac{1}{2} K(1) \ (1+s_o^c) - W \qquad (5)$$

where W is an adjustment for possible capillary rise flux, Bras (1990), which is expressed as:

$$W = K(1) \ (1 + [\frac{3}{2}/(mc - 1)]) \ (\frac{\psi(1)}{z})^{mc} \qquad W < e_p \ , z > \psi(1) \qquad (6)$$

The actual infiltration rate f_i is expressed as:

$$f_i = i \qquad 0 \leq t \leq t_p \qquad i < f_i^*(t,s_i) \qquad (7)$$

and

$$f_i = f_i^*(t,s_i) \qquad t_p \leq t \leq t_r \qquad i > f_i^*(t,s_i) \qquad (8)$$

where i is the gross rainfall intensity, t_p is time to ponding and t_r is the storm duration. The infiltration depth at time t_2 is computed as:

$$INFIL = \int_{t_1}^{t_2} f_i(t) \ dt \qquad t_p \leq t_1 < t_2 \leq t_r \qquad (9)$$

and

$$INFIL = \int_{t_1}^{t_2} i(t) \ dt \qquad 0 \leq t_1 < t_2 \leq t_r \leq t_p \qquad (10)$$

which becomes

$$INFIL = i \times (t_2 - t_1) \qquad 0 \leq t_1 < t_2 \leq t_r \leq t_p \qquad (11)$$

after integration INFIL is as follows:

$$INFIL = \int_{t_1}^{t_2} f_i^*(t,s_i)\, dt \qquad t_p \le t_1 < t_2 \le t_r \tag{12}$$

$$INFIL = S_i \times (t_2^{1/2} - t_1^{1/2}) + A_i \times (t_2 - t_1) \qquad t_p \le t_1 < t_2 \le t_r \tag{13}$$

Philip's expression is derived under the assumption that groundwater table is at infinity. Thus, Philip's infiltration expression may be invalid in very humid and swampy areas because of the shallowness of their groundwater table.

Excess rainfall depth for each time step is computed as the gross rainfall depth minus the total infiltration depth as follows:

$$R_e(j) = R(j) - INFIL \tag{14}$$

Equation 14 illustrates that no excess rainfall occurs unless infiltration losses are satisfied. This excess rainfall will be used for the computation of the overland flow and then the flow hydrograph.

Manning's formula is used to compute the overland flow from each cell to the outlet of its subcatchment which represent a routing node on the stream network of the watershed. To route these flows from each routing node to the next to the watershed outlet the Muskingum-Cunge hydraulic routing method is used. This method can be expressed as:

$$Q(I+1,J+1) = C1 \times Q(i,J+1) + C2 \times Q(I,J) + C3 \times Q(I+1,J) \tag{15}$$

where

$$C1 = \frac{\Delta t - (2 \times TRT(I,J) \times XX)}{2 \times TRT(I,J) \times (1 - XX) + \Delta t} \tag{16}$$

$$C2 = \frac{\Delta t + (2 \times TRT(I,J) \times XX)}{2 \times TRT(I,J) \times (1 - XX) + \Delta t} \tag{17}$$

$$C3 = \frac{(2 \times TRT(I,J) \times (1 - XX)) - \Delta t}{(2 \times TRT(I,J) \times (1 - XX)) + \Delta t} \tag{18}$$

where C1 + C2 + C3 = 1 and

$$XX = 0.5 \times (1 - (\frac{Q(I,J)}{(B(I) + 2 \times Z(I) \times y(I,J)) \times CK(I,J) \times S_o(I) \times L(I)}, \tag{19}$$

$$TRT(I,J) = \frac{L(I)}{CK(I,J)} \tag{20}$$

$$CK(I,J) = \frac{1}{(COEFM(I) \times (WP(I,J))^{2/3}/(S_o(I))^{1/2})^{2/3} \times \beta \times (Q(I,J))^{(\beta - 1)}} \tag{21}$$

DATA REQUIRED

Data required for this model can be categorized as follows:

1. Overland Flow

Overland flow for each cell requires averaged surface layer slope, soil roughness coefficient, and effective rainfall depth. Slope can be obtained from a digitized elevation map. Effective rainfall can be determined from the isoheytal map for gross rainfall minus infiltration losses. Soil roughness coefficient can be estimated from the corresponding values of Manning's coefficient.

2. Infiltration Losses

Philip's infiltration expression data requirements are limited to the characteristics of soil texture and soil structure which can be obtained from soil laboratory analysis.

The pore size distribution index, m, can be obtained from soil laboratory analysis from the following relationship:

$$\psi(s) = \psi(1)\, s^{-1/m} \tag{22}$$

where

$\psi(s)$ matrix potential of soil at soil moisture s [L].

Eagleson (1978c) described pore disconnectedness index, c, as a function of m as follows:

$$c = \frac{(2 + 3m)}{m} \tag{23}$$

and d as a function of c and m as

$$d = c - \frac{1}{m} - 1 \tag{24}$$

The saturated soil matrix potential $\psi(1)$ according to Eagleson (1978c) can be obtained as a function of K(1), poros and the pore shape factor (Φ_{SF}) as follows:

$$\psi(1) = 1.29 \times 10^3 \times \left(\frac{poros}{K(1)\Phi_{SF}}\right)^{\frac{1}{2}} \tag{25}$$

where

Φ_{SF} = pore shape factor.

Eagleson (1978c) expressed Φ_{SF} as a function of m as follows:

$$\Phi_{SF} = 10^{0.66 + \frac{0.55}{m} + \frac{0.14}{m^2}} \tag{26}$$

In order to evaluate Philip's expression, a set of only five parameters are needed. These parameters are s_o, K(1), m, poros, and z. Other parameters can be obtained from K(1) and m as demonstrated in equations above. Diaz-Granados, Bras and Valdes (1983) made regression analysis and obtained an expression for the average soil moisture as follows:

$$\bar{s}_o = 0.276 \times \left(\frac{m_{t_r}}{m_{t_b}}\right) + 0.263 \times \ln\left(\frac{m_i}{\bar{e}_p}\right) + 0.377 \times \left(\frac{\psi(1) \times n}{m_{t_b} \times K(1) \times m}\right)^{1/6} \tag{27}$$

Therefore, pore size distribution index (m), effective saturated hydraulic conductivity (K(1)), effective porosity (n), mean storm duration (m_{t_r}), mean time between storms (m_{tb}), mean storm intensity (m_i), and average potential evaporation rate ($\bar{e}_p$) are the only data required for infiltration losses computations.

3. Routing

Data required for routing process for each stream reach are average bed width, average side slope, average bed slope and reach length.

CASE STUDY

Walnut Gulch watershed is an ephemeral stream located in the San Pedro River basin in southeastern Arizona as shown in Figure 1. The upper portion of Walnut Gulch basin is brush-grass, the lower two thirds of the basin is largely brush-covered.

In this work, a sub-basin of the Walnut Gulch watershed has been selected as the test basin in order to evaluate the hydrologic model. Due to lack of digitized maps of the entire watershed, Walnut Gulch watershed number 11 (WG11) for which DEM's do exist, shown in Figure 2, is selected to calibrate and verify this model. WG11 has significant large-scale rainfall variability (Goodrich, 1990). WG11 rainfall, its corresponding runoff, and initial soil moisture for calibration and verification events are obtained from ARS at Arizona. Two rainfall events are used for calibration and another two events are used for verification.

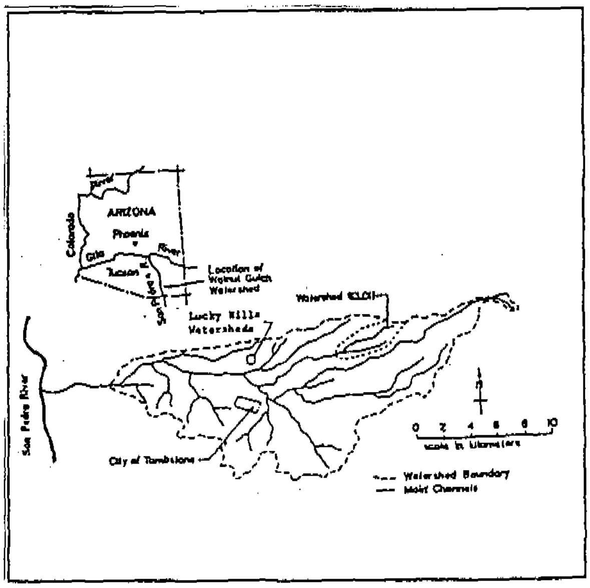

Figure 1. Walnut Gulch Watershed.

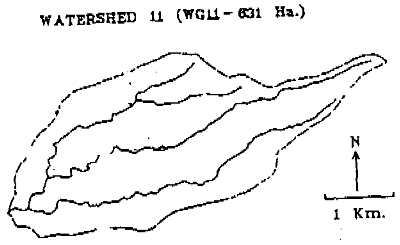

Figure 2. Walnut Gulch Watershed # 11.

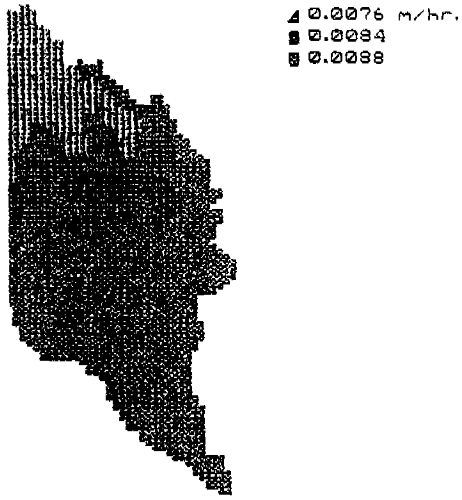

Figure 3. K(1) Distribution Map.

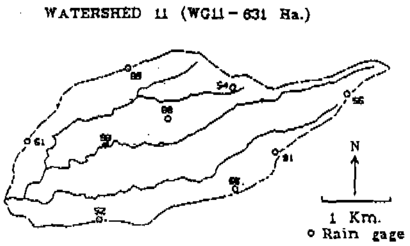

Figure 4. Rain gages Distribution.

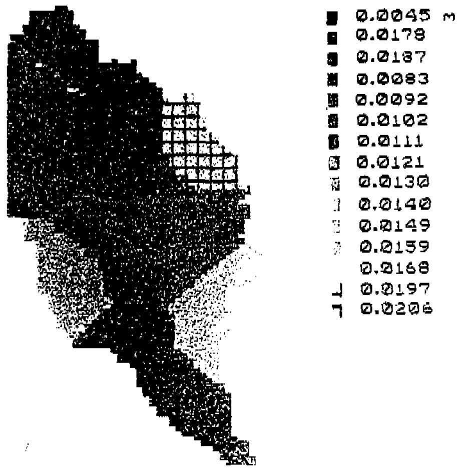

Figure 5. Rainfall Distribution Map.

pMAP is used to reclassify the Soil Map in order to produce the Saturated Hydraulic Conductivity Map, as shown in Figure 3, the Saturated Matrix Potential Map, Porosity Map and Pore Size Distribution Map for each soil type. The values of K(1) and poros were obtained from ARS at Arizona. The value of $\psi(1)$ is computed from K(1) for each soil type. The value of m is selected from previous studies for the same soil types (e.g., Eagleson, 1978c and Allam, 1990).

WG11 has nine rain gages distributed over the entire watershed as shown in Figure 4. Rain gage map is created using pMAP. The initial soil moisture at each rain gage has been obtained from ARS, Arizona. pMAP reclassified the Rain Gage Map in order to generate Initial Soil Moisture Map. Interpolation for the unsampled cells for rainfall and initial soil moisture is performed by pMAP as shown in Figures 5 and 6.

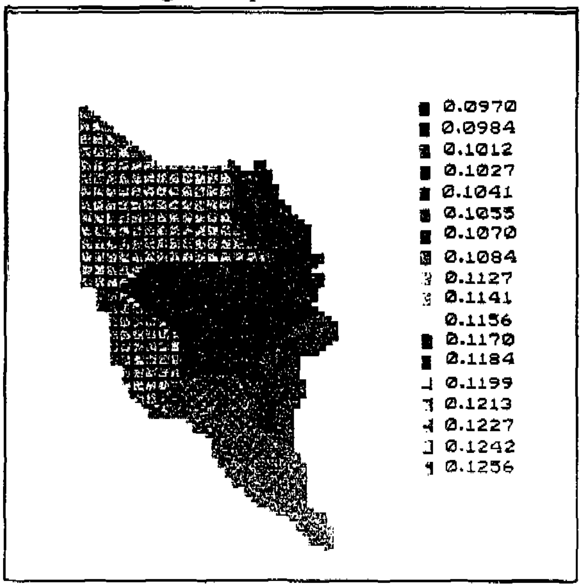

Figure 6. Soil Moisture Distribution.

These generated maps represent the infiltration expression parameters and the total infiltration depth is determined. Rain gage map has been reclassified for generating the rainfall distribution map for each time step. Then, excess rainfall distribution map for each time step, is determined. Any rainfall prior to ponding time is consumed by infiltration losses.

In order to compute the overland flow for each contributing cell, Soil Map is reclassified to develop the Manning's roughness coefficient map. Contributing cells are those cells having non-zero values for MAN, R_c and S_{oc}. The overland flow from these cells is routed to the outlet of each subcatchment. Routing of the overland flow is an additive process to accumulate the flow from each grid cell for each time step. Routing from all routing nodes to WG11 outlet for each time step to produce the discharge hydrograph is performed using the Muskingum-Cunge method.

CALIRATION AND VERIFICATION

WG11 watershed area is 7.85 km^2. The rainfall event of 08-04-1980 and 06-22-1977 are selected to be the calibration events. The computational time step for simulating the watershed response for rainfall is determined as 10 minutes. This selection is made according to the computation of the average overland travel time, TRT_{ov}. This selection has been tested against the average travel time in routing reaches, TR, and it is found reasonable. Thus the selection is made according to the overall basin behavior. Rainfall distribution for the entire watershed is established by interpolating the rainfall depth every Δt at each rain gage. Maps of excess rainfall distribution for each time step are used to determine the overland contribution from each cell to its subcatchment's outlet. Overland contribution from each cell according to the rainfall depth shown in Figure 5 is shown Figure 7.

The model simulated WG11 outlet hydrograph for event of 08-04-80 is shown in Figure 8. The simulated hydrograph peak discharge is 0.12% less than the observed peak discharge. The simulated time to peak exactly matches the observation. Total runoff volume is underestimated by 0.88%.

Rainfall event of 06-22-1977 is the second calibration event. The model simulated WG11 outlet hydrograph for this event is shown in Figure 9. The simulated hydrograph peak discharge is overestimated by 5.28%. The simulated time to peak is overestimated by 33.33%. Total runoff volume is underestimated by 17.31%.

The model has been tested for the rainfall event of 08-04-80 with the same parameters values of the event 06-22-77 without any adjustment. The difference between the observed and the simulated peak discharge is found 32.5% as shown in Figure 10. Time to peak, 20 minutes, exactly matches the observation. Total runoff volume is underestimated by 23.7%. The model has been tested again for the rainfall event of 06-22-77 with the parameters of event 08-04-80, for calibration process, without any adjustment. The simulated peak discharge is overestimated by 54.66%. No improvement has been found in time to peak. Simulated total runoff is 15.30% more than the observed as shown in Figure 11.

Since the cross-sectional geometry and the corresponding Manning's coefficient are estimated for the routing reaches, average values for these parameters are used in order to test both events again. The simulated peak discharge for event 08-04-80 is found 17.28% less than the observed. No change is observed in time to peak. Total runoff is underestimated by 16.02% as shown in Figure 12. The predicted peak discharge for event 06-22-77 is overestimated by 27.53%. There is no improvement to time to peak. Total runoff volume is underestimated by 16.92% as shown in Figure 13.

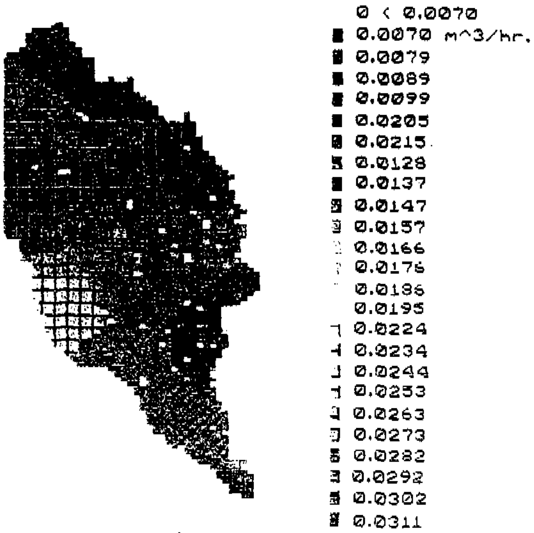

Figure 7. Overland Contribution Map.

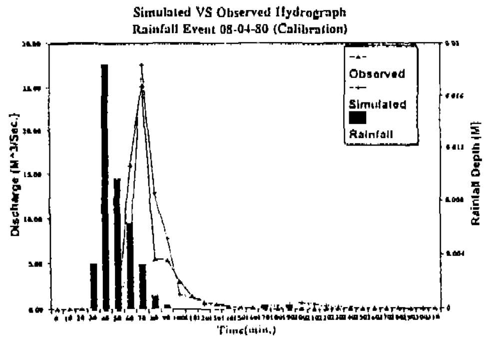

Figure 8. Simulated vs. Observed.

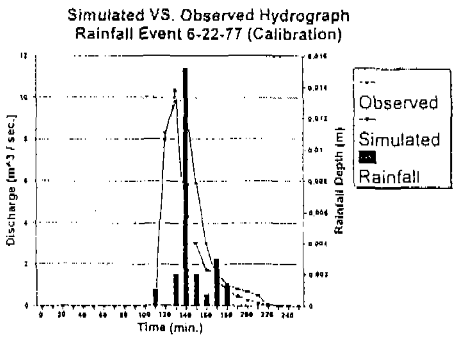

Figure 9. Simulated vs. Observed.

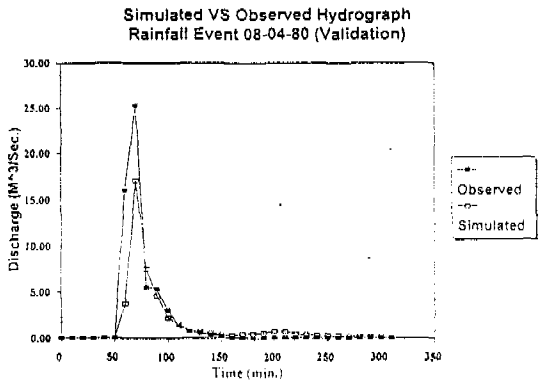

Figure 10. Simulated vs. Observed.

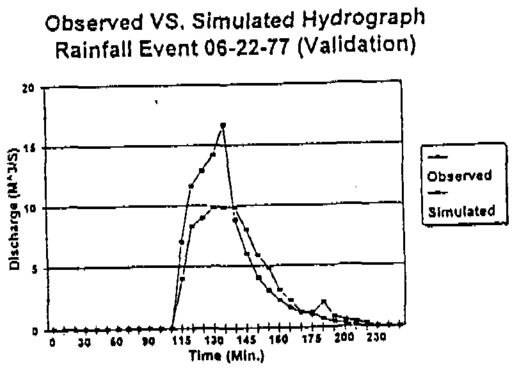

Figure 11. Simulated vs. Observed.

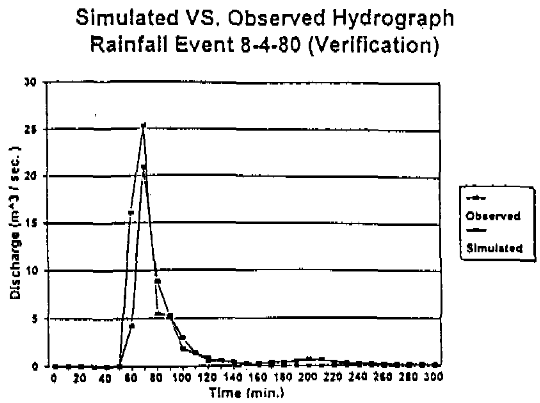

Figure 12. Simulated vs. Observed.

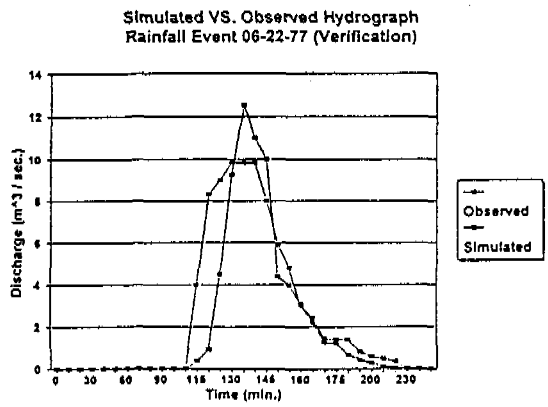

Figure 13. Simulated vs. Observed Hydrograph.

Calibration results produce good performance as shown in Figures 8 and 13. Calibration of the model behavior compares well with similar studies for the same watershed and rainfall events (e.g., Goodrich, 1990). The verification events with no single parameter adjustment compare well with similar previous studies on the same event and watershed (e.g., Goodrich, 1990), while results were not as good as expected. If the problem of estimating the cross-sectional geometry and the corresponding Manning's coefficient for routing reaches is considered, the verification results for both events compares very well with similar studies (e.g., Goodrich, 1990 and Lane, 1982).

Summary and Conclusions

A single event physically-based distributed hydrologic model has been developed. The model has been calibrated for two rainfall events and compares very well with similar studies. GIS has provided its capability for handling large arrays of spatial data. GIS ensured its potentials as programming environment. Most of the hydrologic and hydraulic processes have been evaluated by GIS. Only hydraulic routing has been modelled by FORTRAN. GIS lacks some of the analytic processes which are needed for hydrologic modeling such as iteration and logic statements.

Notation

A_i	gravitational infiltration rate accounting for the water table in fluence $[LT^{-1}]$,
B(I)	averaged bed width for the channel reach,
c	pore disconnectedness index,
COEFM(I)	averaged Manning's coefficient for the channel reach,
d	diffusivity index,
e_p	potential evaporation rate $[LT^{-1}]$,
$\bar{e}_p$	average potential evaporation rate $[LT^{-1}]$,
$f_i^*(t)$	infiltration capacity $[LT^{-1}]$,
INFIL	infiltration depth at time t_2 (L),
K(1)	saturated hydraulic conductivity $[LT^{-1}]$,
L(I)	channel reach length,
$\psi(1)$	saturated matrix potential of soil [L],
m	pore size distribution index,
m_{t_r}	mean storm duration [T],
m_{t_b}	mean time between storms [T],
m_i	mean storm intensity $[LT^{-1}]$,
poros	effective soil porosity,

R(j)	gross rainfall depth at time step j [L],
R_e(j)	excess rainfall depth at time step j [L],
s_i	initial soil moisture in the surface boundary layer,
S_i	infiltration sorptivity [$LT^{-1/2}$],
So(I)	averaged bed slope of the channel reach [L/L],
W	apparent velocity of capillary rise from water table [LT^{-1}],
WP(I,J)	averaged wetted perimeter of the channel reach,
y(I,J)	corresponding flow depth,
z	depth to groundwater table [L], and
Z(I)	averaged side slope for the channel reach.

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MODELING IRRIGATION AND DRAINAGE MANAGEMENT IN THE PRESENCE OF WATER TABLE

Farida El-Hessy [1]

ABSTRACT

A numerical model was developed to be used as a tool for irrigation and drainage system design and management in areas with saline and shallow groundwater table. The model computes spatial and temporal distribution of soil moisture and salinity as affected by field-scale practices of irrigation and drainage in the presence of such water table. In addition to calculating salinity and water distributions, the model predicts depth to the water table, upward flux from the water table, leaching efficiency, volume and salinity of drainage effluent, and relative crop yield. The model explicitly considers variability due to the diverse properties and irrigation practice on multiple fields in one area.

MODELAGE DE LA GESTION D'IRRIGATION ET DE DRAINAGE EN PRÉSENCE DE L'EAU SOUTERRAINE

RÉSUMÉ

Un modèle numérique a été développé pour être utilisé au cours de la conception et de la gestion des systèmes d'irrigation et de drainage dans les régions où il y a de l'eau souterraine saline et superficielle. Le modèle calcule la distribution spatiale et temporelle de l'humidité et de la salinité du sol quand ce dernier est affecté par la pratique d'irrigation et de drainage au niveau du champ en présence de telle qualité d'eau souterraine. En plus du calcul de la distribution de salinité et d'humidité, le modèle prédit la profondeur de l'eau souterraine, l'efficacité du lavement du sol, le volume et la salinité des effluents de drainage et le rendement relatif des cultures. Le modèle considère explicitement la variabilité causée par les diverses propriétés et la pratique d'irrigation aux champs multiples de la même région.

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1. BACKGROUND

Many field studies have been carried out to evaluate the contribution from saline groundwater to crop requirement (Grimes 1984, and Kruse et al. 1984, 1985, 1986). Torres (1987), and Torres and Hanks (1989) used a numerical model to predict the contribution. Nour el-Din (1986), and Nour el-Din et al. (1987) studied the effect of different strategies on salt concentration profile using a numerical model. Empirical models are in use to predict the impact of saline shallow water table on crop yield (Grismer et al., 1988; and Gates, 1988).

Transient flow through saturated-unsaturated porous media have been described by two different approaches, Chung (1987):
1) Linked models which admit a fundamental difference in the flow in the saturated zone and that in the unsaturated zone, and attempt to model the two systems separately.
2) Composite models which consider that flow exhibits sufficient continuity across the water table so that water flows continuously irrespective of whether it is above or below the water table in the whole soil-water-air system. Hence, it is mathematically unnecessary to differentiate between the saturated and unsaturated zones.

2. MODEL DEVELOPMENT

The approach of linked models is used in this study because it allows solving the saturated flow equation in the horizontal plane which is needed to simulate a variety of conditions in different farms.

2.1 Water Movement in Soil

2.1.1 Unsaturated submodel

The unsaturated flow domain is divided into several one-dimensional vertical columns, each column represents one grid. These columns are modeled by solving the capillary pressure form of the Richard's equation which can be expressed as:

$$\frac{\delta\theta}{\delta t} = \frac{\delta}{\delta z}[k(h_c)\frac{\delta h_c}{\delta z}] + \frac{\delta k(h_c)}{\delta z} + S \qquad (1)$$

where:

θ = volumetric moisture content (L^3/L^3),
t = time (T),
z = depth (L),
h_c = capillary pressure head (L),
$k(h_c)$ = unsaturated hydraulic conductivity as a function of the capillary pressure head

(L/T), and

S = sink term representing volumetric water extraction by plant roots (L^3/L^3) .

The unsaturated column is divided into a number of vertical nodes. An implicit finite difference approach is used to solve the matrix obtained from the partial derivatives of these equations which is solved for the change in pressure head at each node(Jalut; 1989).

The root water uptake model used in the present study was that developed by Van Genuchten (1987):

$$S = \alpha(h,\pi)\, S_m \qquad (2)$$

where:

S = actual extraction rate (L/T),
h = soil water suction ,
π = osmotic head ,
$\alpha(h,\pi)$ = a dimensionless water and salinity stress response function, and
S_m = the maximum (potential) extraction rate (L/T).

The general form of the total stress response function is:

$$\alpha(h,\pi) = \frac{1}{1+\left[\frac{(a_1 h + a_2 \pi)}{h_{50}}\right]^p} \qquad (3)$$

where

h_{50} = the suction head at which extraction is reduced by 50 % .
a_1, a_2 = weight of soil water pressure and osmotic head on water uptake, and
p = an empirical constant.

note that $\alpha(h,\pi)=0.5$ when $p=0.0$ and/or $a_1 h + a_2 \pi = h_{50}$.

The following form was suggested for the maximum extraction term S_m,

$$S_m(z) = T_p\, \lambda(z) \qquad (4)$$

where

T_p = potential transpiration, and
λ = depth-dependent root distribution function which is given by

$$\lambda = \frac{-1.6}{d^2} z + \frac{1.8}{d} \quad (5)$$

z = vertical distance positive downward (L).

where
d = vertical depth of the root zone (L), and This equation gives the average extraction pattern which reflects the distribution of active roots in normal soil conditions with the largest concentration of roots near the soil surface. It fits the empirical rule of root extraction i.e. 40,30,20 and 10 percent of total transpiration requirement supplied by each successive quarter of the root zone.

2.1.2 Saturated submodel

The nonlinear partial differential equation describing transient, two-dimensional flow in a saturated unconfined porous medium may be derived from the mass continuity equation and Darcy's law and may be written as:

$$\frac{\delta}{\delta x}[k_x h \, \Delta y \frac{\delta H}{\delta x}] \, \Delta x + \frac{\delta}{\delta y}[k_y h \, \Delta x \frac{\delta H}{\delta y}] \Delta y = Q + S \, \Delta x \, \Delta y \, \frac{\delta H}{\delta t} \quad (6)$$

where

K = hydraulic conductivity (L/T),
h = saturated thickness of aquifer (L),
H = water table elevation above a datum (L),
S = storage coefficient,
Q = net volume of water added or withdrawn from the aquifer
$\Delta x, \Delta y$ = horizontal space dimensions(L), and
t = time (T).

Equation (6) is nonlinear and cannot be solved for the general case. Applying numerical techniques, dividing the region of flow into a grid system and using an implicit central finite difference scheme, Equation (6) can be written for one of these grids for a particular time step and the entire system of equations is solved simultaneously. The solution will give predicted water table elevation.

2.1.3. Upward flux

The model calculates upward flux for each grid as the negative flow in the lower node of the unsaturated column at each time step. The cumulative upward flux is also calculated from the beginning of simulation until the time step under consideration.

2.2 Salt Transport

2.2.1 Unsaturated zone

Salt transport in soils can be described by the advective-diffusion equation . In a partially saturated zone, salt transport due to dispersion is negligible compared to the convective. The mixing cell (mass balance) approach is used to get salt concentration for each node as follows:

Mass in - Mass out = Change in mass stored

$$C_{d-1}^{t}\, q_{d-1}\, \Delta t - C_{d+1}^{t}\, q_{d+1}\, \Delta t = C_{d}^{t}\, \theta^{t}\, \Delta z_{d} - C_{d}^{t+\Delta t}\, \theta^{t+\Delta t}\, \Delta z_{d} \qquad (7)$$

where:

C_d^t = salt concentration in node (d) at time (t) (M/L^3),
q_d^t = water flux out of node(d) at time (t) (L/T),
θ_d^t = moisture content of node (d) at time (t) (L^3/L^3),and
Δz_d = vertical dimension of the node (d) at time (t) (L).

2.2.2 Saturated zone

Each finite difference grid in the ground-water aquifer is considered a single cell in which a complete mixing of salts occurs during each time step. Therefore, the salt concentration does not change with depth. With the previous assumption, the advection dominated conservative salt transport equation for two-dimensional flow is given by:

$$-\frac{\delta C}{\delta t} = \frac{\delta C\ V_x}{\delta x} + \frac{\delta C\ V_y}{\delta y} + S \qquad (8)$$

where:

C = salt concentration of ground water (M/L^3),
t = time (T),
x,y = dimensions for horizontal plane (L),
V = volumetric flux, determined from the flow equation
= q/ϕ, and
S = source/sink term (amount of salts added or removed from the aquifer).

Equation (8) is solved for $C_{x,y,t}$ explicitly using the mixing cell approach.

3. IRRIGATION SCHEDULE

3.1 Applieed Water

The model schedules irrigation according to the deficit of moisture in the root zone, which is calculated. When irrigation time is reached (from input data), it calculates the required depth to be applied from the calculated deficit in moisture and given irrigation application efficiency. The application time for an irrigation is calculated by dividing the applied depth by the rate of application. Then the time of application is taken equal to the next higher integer number of time steps and the rate of application is modified to be applied for these time steps as follows:

$$Q_{ap} = \frac{d}{N * \Delta t} \tag{9}$$

where:

Q_{ap} = real application rate for irrigation (L/T) ,
d = depth of applied water (L),
Δt = time step (T), and
N = number of time steps during which irrigation take place.

$$N = T_{ap} / \Delta t \tag{10}$$

$$T_{ap} = d/Q \tag{11}$$

where:

T_{ap} = time of application (T), and
Q = irrigation application rate (input data) (L/T).

$$d = \frac{TMD}{E_a} \tag{12}$$

where:

TMD = total moisture deficit in the root depth (L), and
E_a = application efficiency.

The model provide constraint to the maximum depth of applied water according to the practical values in the area to be simulated.

The model has the capability of using different parameters (like date of first irrigation, application rate, and irrigation application efficiency) for different fields. Due to the uncertainty of these parameters, the model helps to adopt the distribution which better simulates the real situation of the area under study.

3.2 Crop Water Requirements

The potential evapotranspiration is an input data to the model, which could be measured from lysimeters or predicted from one of the available models like, Penman, Blaney-Criddle, and Jensen-Haise. The actual evapotranspiration is calculated as follows:

$$ET_a = \propto K_c \; ET_p \qquad (13)$$

where:

ET_a = actual evapotranspiration (L/T),
$\propto$ = water and salinity stress response function,
K_c = crop coefficient, and
ET_p = potential evapotranspiration (L/T).

The model calculates daily crop coefficient based on the following input data: date of planting, date of beginning and end of each stage (namely crop development, mid-season), harvest date, crop coefficient at planting, mid season and harvest.

3.3 Root Development

The root depth is calculated daily by proportion from the maximum as follows:

$$RD = \frac{RDM - RDI}{DI - DP} (D - DP) \qquad (14)$$

where:

RD = root depth at day D (L),
RDM = maximum root depth (L),
RDI = initial root depth (L),
DI = day of reaching maximum root depth (Julian day),
DP = planting day (Julian day), and
D = present day (Julian day).

4. RELATIVE YIELD

The Relative Yield (Gates and Grismer, 1989) is defined as the ratio of crop yield occurring as the result of a given combination of available water, soil water salinity and water table depth to the potential yield that would be expected if no such effects were present. The relative yield could be expressed as

$$RY = RY_{ws} \, RY_D \qquad (15)$$

where:

RY = relative crop yield as a function of the combined effect of available water, soil water salinity and water table depth,

RY_{ws} = relative crop yield as a function of available water and soil salinity, and

RY_D = relative crop yield as a function of depth to a relatively nonsaline water table.

The effect of matric and osmotic stress on yield may be quantified by relating the relative yield decrease to the relative deficit in evapotranspiration. Doorenbos and Kassam (1979) gave the following equation:

$$(1 - RY_{ws}) = K_y \;\; [1 - \frac{ET_a}{ET_m}] \tag{16}$$

where:

K_y = yield response factor,
ET_a = actual evapotranspiration (L), and
ET_m = maximum potential evapotranspiration (L).

The yield response factor K_y relates relative yield decrease to relative evapotranspiration deficit. Moisture and salinity stress affect plants at all stages of development. According to crop, sensitivity varies from one growth stage to the next. Doorenbos and Kassam (1979) estimated values for K_y for different crops for the total growing period and for individual growth periods.

The effect of depth to water table on yield is quantified by using the average depth to water table. Gates and Grismer (1989) analyzed data for cotton yield in relation to depth to a nonsaline water table in clay and silty clay soils. A linear equation in the following form fitted the data

$$RY_D = a + bd \tag{17}$$

where:
a,b = regression coefficients equals 0.313 and 0.778 respectively , and
d = average seasonal water table depth (m).

5. DRAINAGE

5.1 Leaching Efficiency

The leaching efficiency can be calculated as:
where:
E_l = leaching efficiency,

$$E_l = \frac{C_d - C_i}{C_s - C_i} \tag{18}$$

C_d = salt concentration of the water drained from the root zone,
C_i = salt concentration of the irrigation water applied, and
C_s = salt concentration of the soil water at the upper end of the soil-water content range, taken at field capacity.

5.2 Drainage Effluent

Calculation of water and salt balance for each grid to all directions (in the horizontal plane) enables to calculate the amount of water and salt drained to any specific defined boundary considered such as drains.

6. BOUNDARY CONDITIONS

The upper boundary for the unsaturated zone is the ground surface. The condition on the upper boundary after irrigation is considered as a constant head equal to the depth of ponded water over the ground surface. Each time step this depth of water is reduced by the infiltrated amount and the remaining ponded water is the new constant head for the next time step. This boundary adjustment process continuous until the whole depth of water is infiltrated. Then the upper boundary is treated as a no flux boundary. The lower boundary of the unsaturated zone is the water table, which has constant pressure head equal to atmospheric pressure. The unsaturated column is divided into a number of equal vertical nodes, the upper node is assigned small depth to better simulate this boundary. The boundaries for the saturated zone are either constant head grids which represents canal or drain and assigned a constant value for the whole simulation time or no flow grids which represents impervious boundary and assigned zero transmissivities during the simulation time.

Drains provided inside the area are defined at the beginning as constant head grids, which could be atmospheric pressure to represent free flow of water or specified head to represent submergence of the drain.

7. MODEL VERIFICATION

7.1 Materials and Methods

Data from Kool and Van Genuchten(1989) are used to compare the moisture content profile in case of long term drainage(100 days).

Drainage for a long period compared with Kool's model and experimental data. The experimental study was conducted at Los Almos National Laboratory. Transient infiltration and drainage of water in a large caisson was studied. The caisson measured 6 m in depth by 3m in diameter. Neutron probe access tubes and tensiometers were used to measure water content and soil water potentials at different depths.

The initially dry soil in the caisson was first saturated with water and then allowed to drain under gravity. Drainage was simulated using a zero flux boundary condition at the caisson surface and a zero pressure head condition at the base of the caisson.

7.2 Results and Discussions

The simulation was conducted for a period of 1OO days. Predicted moisture profiles in the caisson after 1,4,2O and 1OO days of drainage are compared to measured water content data and to kool and Van Genuchten's results as shown in figure (1). Deviation of the predicted results by the model from experimental data ranges from -1.88% to 7.64% with an average of O.12% .

8. SUMMARY AND CONCLUSIONS

Decision making in irrigation management and drainage system design requires the consideration of many factors which include: crop type, ground-water depth and salinity, salinity of irrigation water, and soil characteristics.

This research is designed to study the effect of a selected combination of irrigation management and drainage system characteristics on the soil salinity, water table depth, and ground-water salinity. Relative crop yield was considered to be the control parameter for evaluating the impact of the chosen irrigation and drainage policy under this condition. The relative yield is controlled by waterlogging, matric stress and osmotic stress.

A numerical model was developed to predict flow and salt transport for the saturated and unsaturated zones and the depth to the water table in the presence of subsurface drains. A saturated-zone ground-water model developed at Colorado State University which solves the depth-averaged Boussinesq equation by the finite difference method was linked to a finite difference solution of Richard's equation for the unsaturated zone . The mass balance approach (mixing cell) was used to solve the advection-dominated, conservative salt transport equation in both the unsaturated and saturated zones.

The model can be used for long term prediction of moisture content, salinity of the unsaturated zone, the contribution from groundwater table to water requirement and drainage for long time simulation.

The mass balance error in the unsaturated zone can be attributed to the sensitivity of the unsaturated flow model to several factors such as: the parameters of the model used for the matric potential-water content relation; and hydraulic conductivity-water content relation e.g. soil index and displacement pressure, the parameters of model used for the effect of matric and osmotic stress on water uptake by plants, and the size of the time and space increment. Also, adjusting the unsaturated column length each time step could affect the accuracy of results.

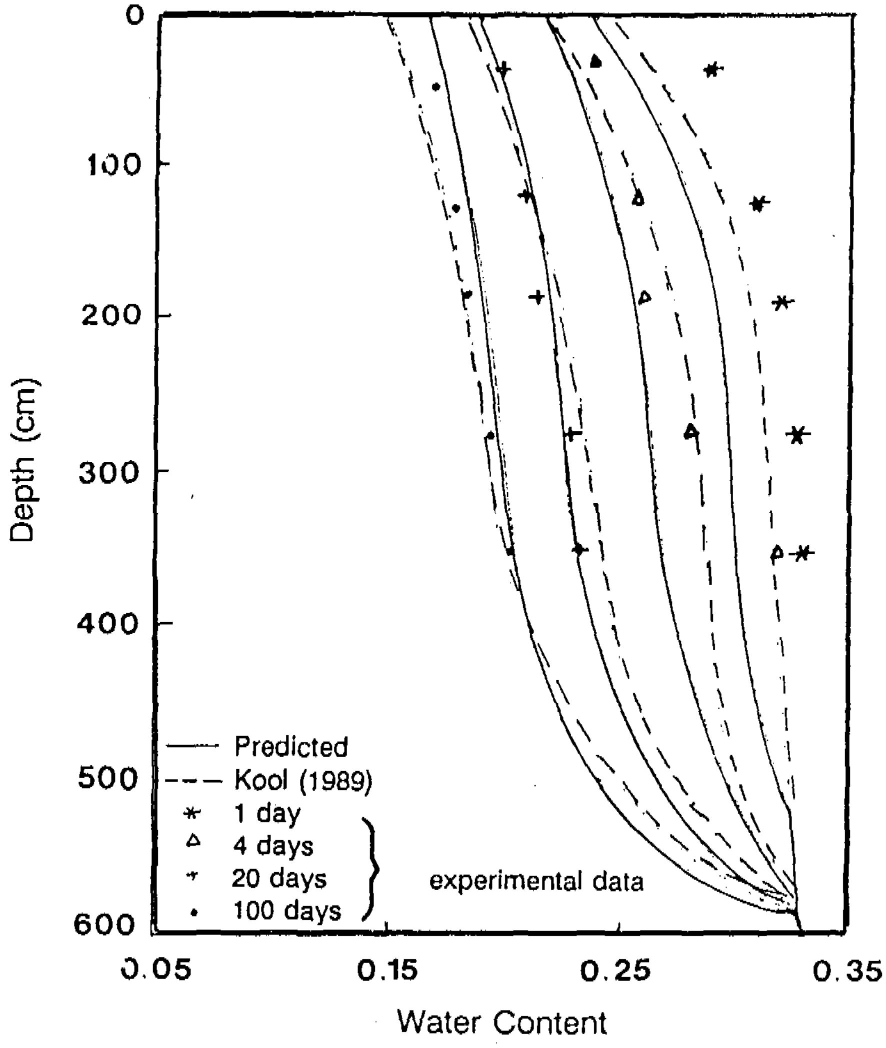

Figure 1. Predicted,Kool's and observed water content profiles during drainage

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ESTIMATION OF LOCALIZED RECHARGE TO UNCONFINED AQUIFER, KLUANG AREA, MALAYSIA

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1. ABSTRACT

Natural recharge refers to the amount of soil moisture content reaching the water table, is the primary mechanism for replenishing the groundwater reservoir. Rainfall is the principal factor for recharge resulting in occurence and movement of moisture in the unsaturated soil water system and fluctuation of the water table elevations. The present study was undertaken to construct a mathematical model and to investigate the recharge mechanism in unsaturated zone to a water table using an approximate solution (Green and Ampt model). The response of the water table to a localized natural recharge is solved using Bousinesq's equation. The model is used to simulate experimental and field condition as nearly as possible. The results obtained from the experimental and field data were compared with the numerical analysis. The comparison revealed that a good agreement achieved between the experimental and field results against computed analysis.

2. INTRODUCTION

The government of Malaysia, as part of its program for providing adequate infrastructure services to nation, has formulated the Kluang Water Supply Project in order to propose a scheme to meet the predicted domestic and industrial requirement of Kluang District up to year 2010. The first stage implementation is identified to meet the needs of the project area up to the year 1995, location of the project area is shown in Figure 1 (Kluang New Water supply Project, Feasibility study report, 1983). The major sources of natural recharge to unconfined aquifer is a direct rainfall infiltration on basin area and/or downward percolation of stream runoff. However, this mechanism can be visualized approximately using a sand tank model. In addition, a field study carried in Kluang area, Malaysia. The groundwater resources was investigated to evaluate the potential of groundwater as a supplementary source for the Kluang water supply. Construction of wells were carried out to abstract groundwater from the Kahang river flood plain to supply

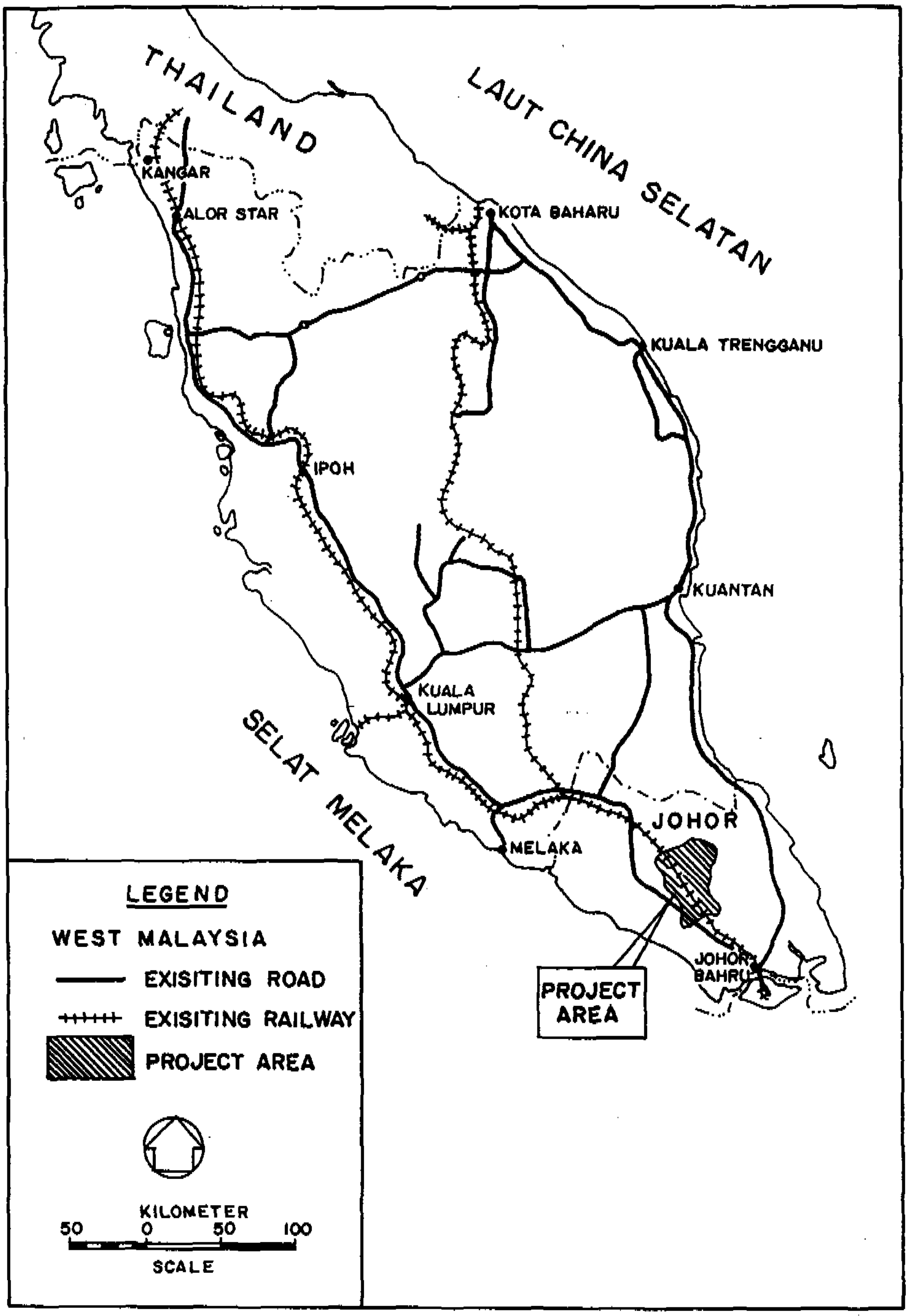


Figure 1 Location Plan of Project Area

water to Kahang area. A pilot study area of size approximately 2 sq. km has been chosen at the right bank of Kahang river area and the location of the pilot area is shown on Figure 2. The published literature could be summarized. Maasland and Shery (1967) concluded that Dupuit Forchheimer assumption were valid for approximate solution of flow between drains spacing was large in comparison to the depth of the aquifer. Marino (1967) investigated the growth and decay of groundwater ridges in aquifers and a good agreement was obtained with rising water table larger than the initial depth of saturation. Cook and Kilty (1992) used electromagnetic method to estimate the recharge and achieved a good results over large area with a little loss of accuracy. Baumann (1952), Maasland (1959), Hantush (1967), Marino (1964,1974a,b,c), W. Mokhtar (1983) and Rai and Singh (1981) used analytical expression to approximate the water table response beneath recharge sites.

3. MATHEMATICAL MODEL

A mathematical model is developed to simulate the problem of the behavior of an unconfined aquifer based on the Green and Ampt approach and a finite difference solution of the Boussinesq's flow equation.

3.1 The Approximate Solution (The Green and Ampt approach)

It is assumed that a homogeneous profile of porous material is saturated, with the water table initially positioned at the lower boundary of the profile. The vertical ordinate z is assumed positively upwards and z=0 at the soil surface. The saturated conductivity is k_{sat} and the soil water pressure which the sharp draining front becomes established is h_f. The difference in volumetric water content between the saturated and drained condition is $\Delta\Theta$. The water table at t=0 starts to fall and rise at a constant rate v. For nonzero applied flux case, the final solution of equation is

$$t= -z/\Omega + h_f /(v+\Omega) \ln \{1+ [(v+\Omega) (vt-z)]\} / (vL -\Omega h_f + \Omega L) \quad 1$$

where $\Omega = k_{sat} / \Delta \Theta$

3.2 Sarurated Flow System

A mathematical model that incorporate the water table response to natural recharge in an unconfined aquifer consists of a governing equation and boundary conditions which simulate

(T6-S2) 4.4

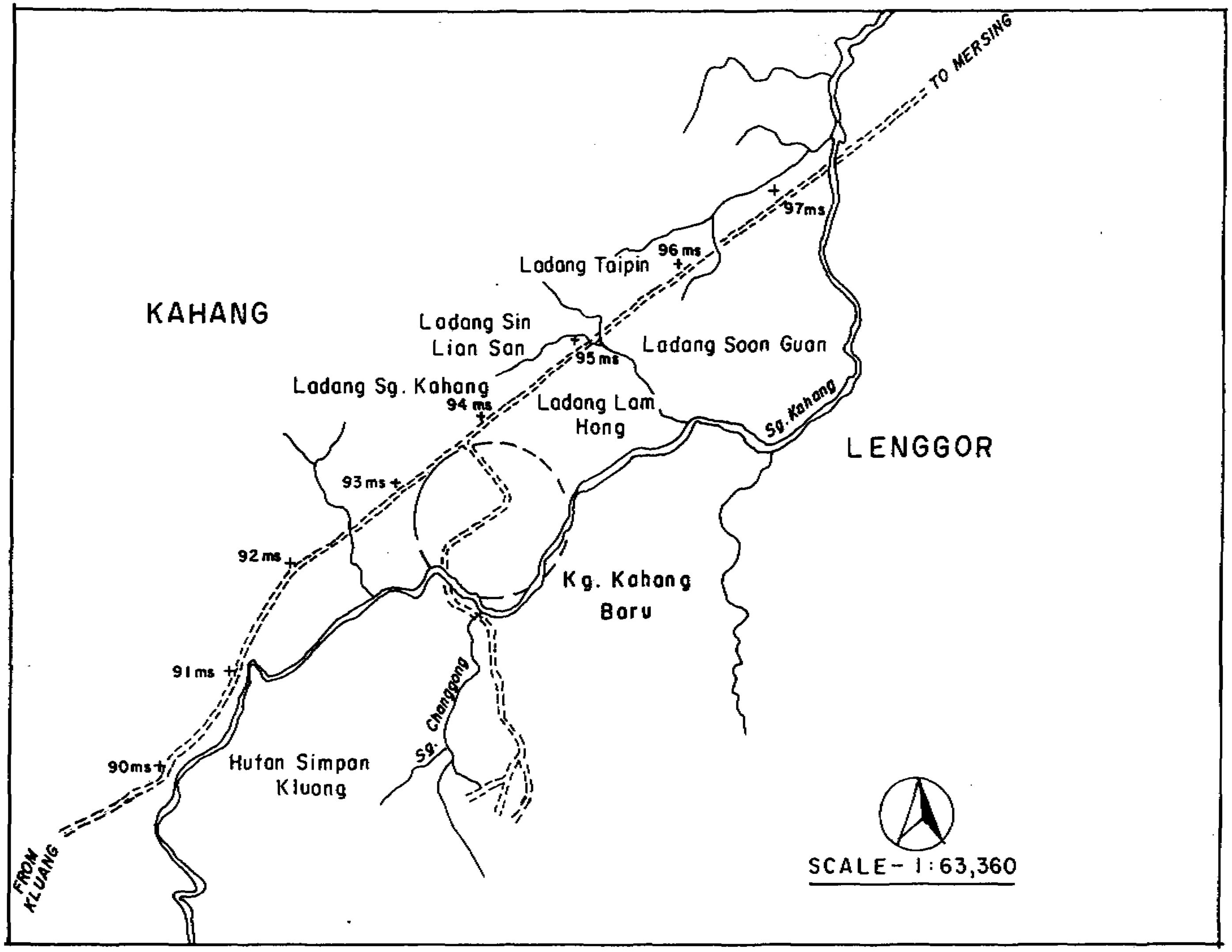


Figure 2 Site Plan of Study Area

the flow of groundwater in particular problem domain. The model requires in a particular calculating values of piezometric heads at each point in the system. This in turn helps in drawing conclusions about the configuration of groundwater flow systems. The governing equation presented generally follows that suggested by Wang and Anderson (1982). Thus, for one dimension, unconfined-unsteady flow equation,

$$k/2\ \partial^2 h / \partial x^2 + R(x,t) = S\ \partial h / \partial t \qquad 2$$

where

R is the recharge rate into the top boundary,

k hydraulic conductivity

S storage coefficient.

h hydraulic head.

In order to solve the flow equarion, ir is necessary to define the boundaries of the numerical schemes. Boundary conditions for the numerical solution are the same as those used in the model. upper and lower boundaries are known aand held constant throughout each run.

$$x=0\ ;\ H(0,t) = H_U$$

$$x=L\ ;\ H(L,t) = H_L$$

in which L is the length of the unconfined aquifer, H_U and H_L are the upper and lower boundaries, respectively. H_U and H_L are the depth of water table in the channels.

To determine the initial condition or steady state values of the free surface, it can be calculated based on Dupuit's assumptions as

$$\frac{d^2\ h^2(x)}{d\ x^2} = 0 \qquad 3$$

Thus, the solution for equation 3 is,

$$h^2(x) = H^2_U + (H^2_L - H^2_U)(x/L) \qquad 4$$

where

L is the length of the unconfined aquifer

H_U and H_L are the depth of water table in the channels.

4. EXPERIMENTAL STUDIES

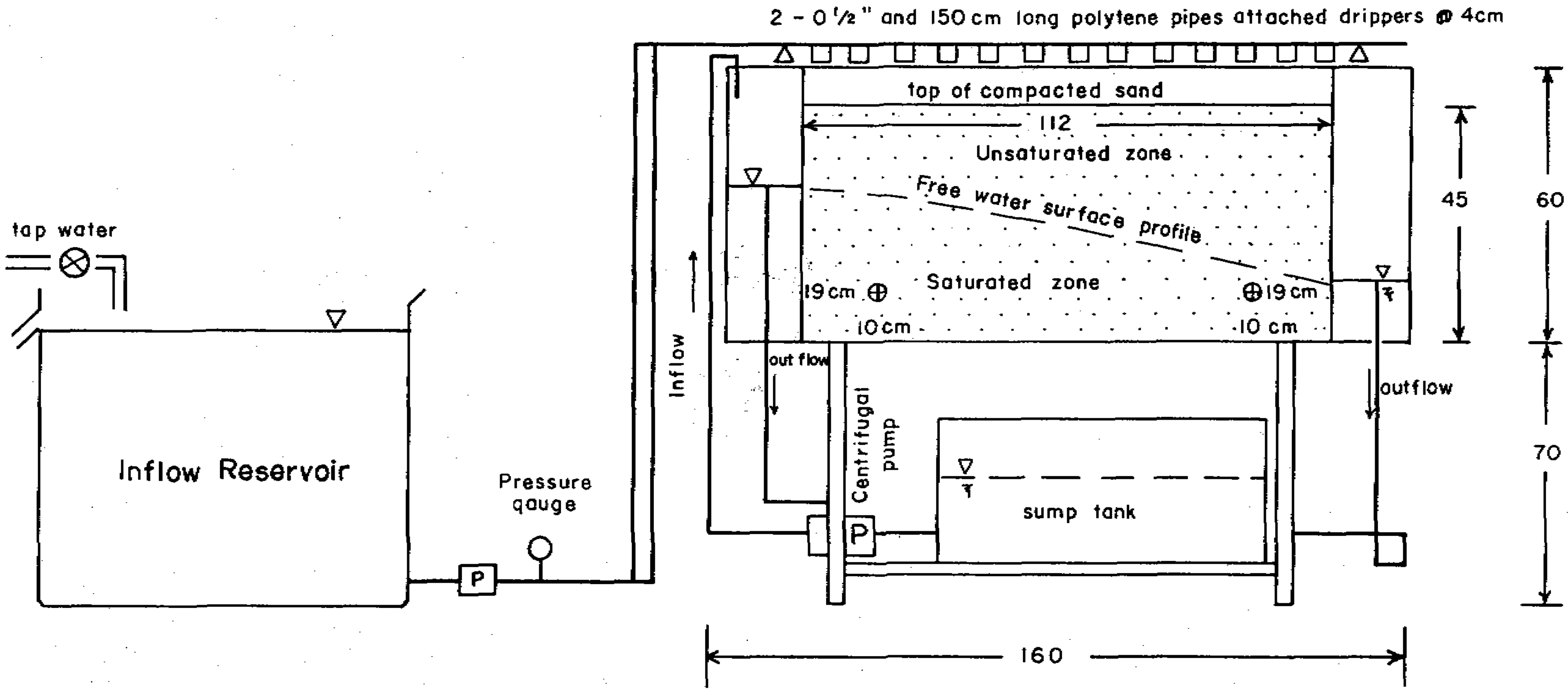
4.1 Laboratory Experiments

The experimental work was performed with a drainage and seepage Tank Model, more commonly known as a sand tank model. The model was constructed to simulate an unconfined aquifer with the assumption of horizontal impermeable layer as datum and coincides with the base of two streams. The upper and lower boundaries are assumed to be permeable and the aquifer is isotropic and homogeneous. The model is subjected to varying boundaries conditions and different rates of recharge on the groundwater surface profile. The sand used for this experiment was a beach sand from Port Dickson with fraction range between 1.18 mm and .6 mm. The capillary rise was -11.7 cm and saturated hydraulic conductivity -14.7 cm/min. The saturated water content was .395. The unconfined aquifer was modelled with the water table surveyed at both sides as the upper and lower boundaries. In order to separate those boundaries from the body of aquifer, the vertical sheets of perforated metal, which was supported by the special designed hollow-steel supporters, were used as shown in Figure 3.

4.2 Field Tests

The geology of the study area is diversified, consisting of several rock types varying in age from permain to recent. The Blumeut granite consisting of granite and adamellite, the south-east half of the area, with a granite massive occupying the north-western sector of the area. Several sedimentary formations are located between the two granitic bodies. The basic geological pattern records alluvial deposits in Kahang area with panti formation further upstream in the Kahang flood river plain. This alluvium consists mainly of coarse sedimentary sands and gravel. Aquifer thickness vary between 5 to 26 feet. Below the aquifer, there was weather rock and natural rock formation. The rainfall/ water table level measurements were made during summer of the year 1988 at a borehole BH8 at Kahang town. The average rainfall was 2300 mm and the water table levels was found to fluctuate between .6 to 1.2 meters shown in Figure 4.

(T6-S2) 4.7



(⊕ Selected tapping points)

Figure 3 Major dimensions and schematic of water flow diagrams for sand - tank model.

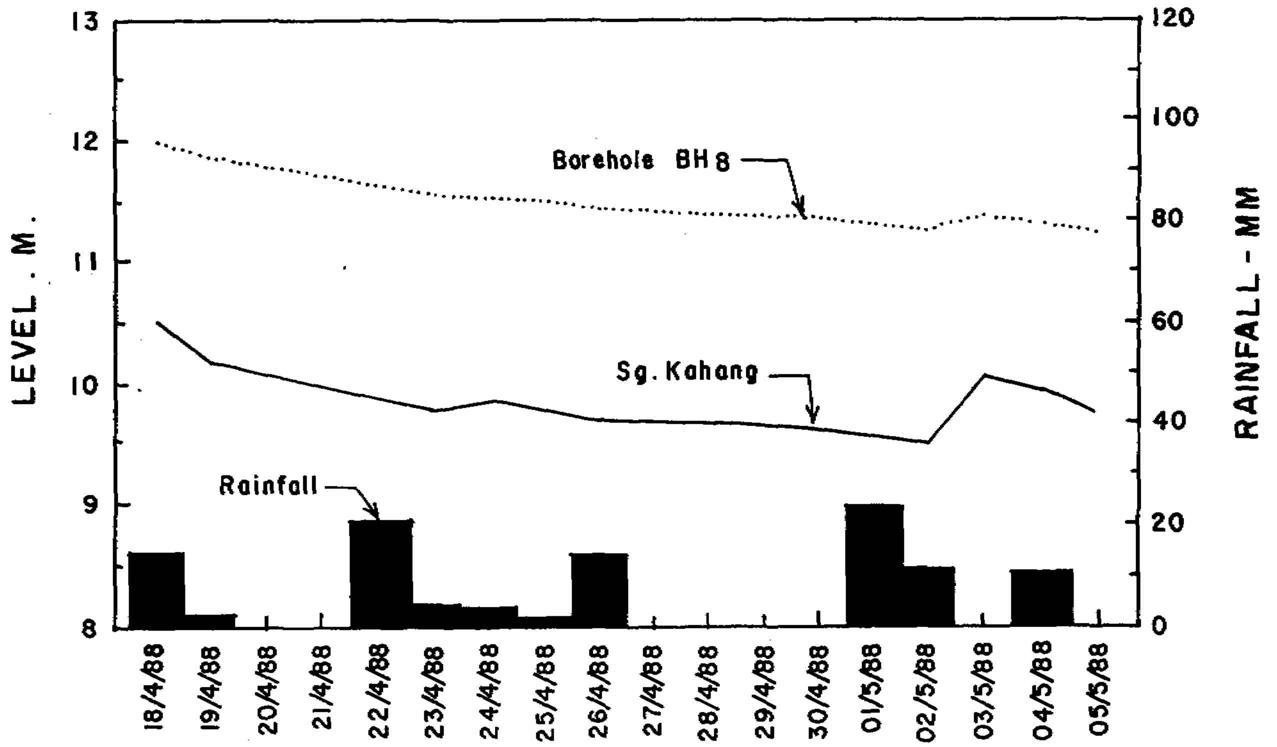


Figure 4 Stage and ground-water table hydrograph, Sg. Kahang at Kg. Kahang Baru, Kluang, JOHOR.

5. RESULTS AND DISCUSSION

The mathematical model to simulate the response of water table to natural recharge is given by equations 1 and 2. The versatility of the model can be verified with an experimental test. For each test, a steady-state flow condition was first established by adjusting outflow tubes. The time needed to reach steady -state condition was about 1 hour. The free surface level was recorded from the two piezometer readings and computed by equation 4. The results obtained showing the relation of water table elevation with time is given in Figure 5. It can be seen from the comparison between the two sets of data a good agreement was achieved. The performance of the integrated model can be noted in Figure 6, solid line and the triangle-dashed line with label "t=0" is the initial free water surface profile resulting from the of both saturated and unsaturated equations and experimental observations respectively.

The application of the Green and Ampt equations involving a rising water to simulate unsteady recharge is shown by graphs labelled "RISING WATER TABLE" while the assumption of the Green and Ampt approach to the drainage regime under the influence of falling water table is shown by the graph labelled" FALLING WATER TABLE". The experimental values lie above the computed values because the stream lines of the mathematical model are all assumed horizontal when actually there is some curvature caused by vertical and horizontal components of the velocities. In addition, the validity of the integrated model has been applied in the field. The comparison between the monitored field and predictive water levels can be seen in Figure 6. A fair agreement exists between the predictive levels and the monitored levels. The integrated model gives an overall lower values than monitored levels. However, it can be seen that the trend in the rise and fall of water levels is similar between the predicted and observed values, Figure 7.

6. CONCLUSION

The model developed in this study provides a new efficient approach to simulate the actual dynamic water table behavior.The application of the Green and Ampt model to simulate the recharge through the unsaturated zone proved to be a simple technique under various moving water table condition. The linearized form of the Boussinesq's equation has been analyzed and found to give a fair approximation of the water table profiles. It can be concluded that the model is suitable for the study of dynamic water table behavior involving transient groundwater flow in a permeable sand.

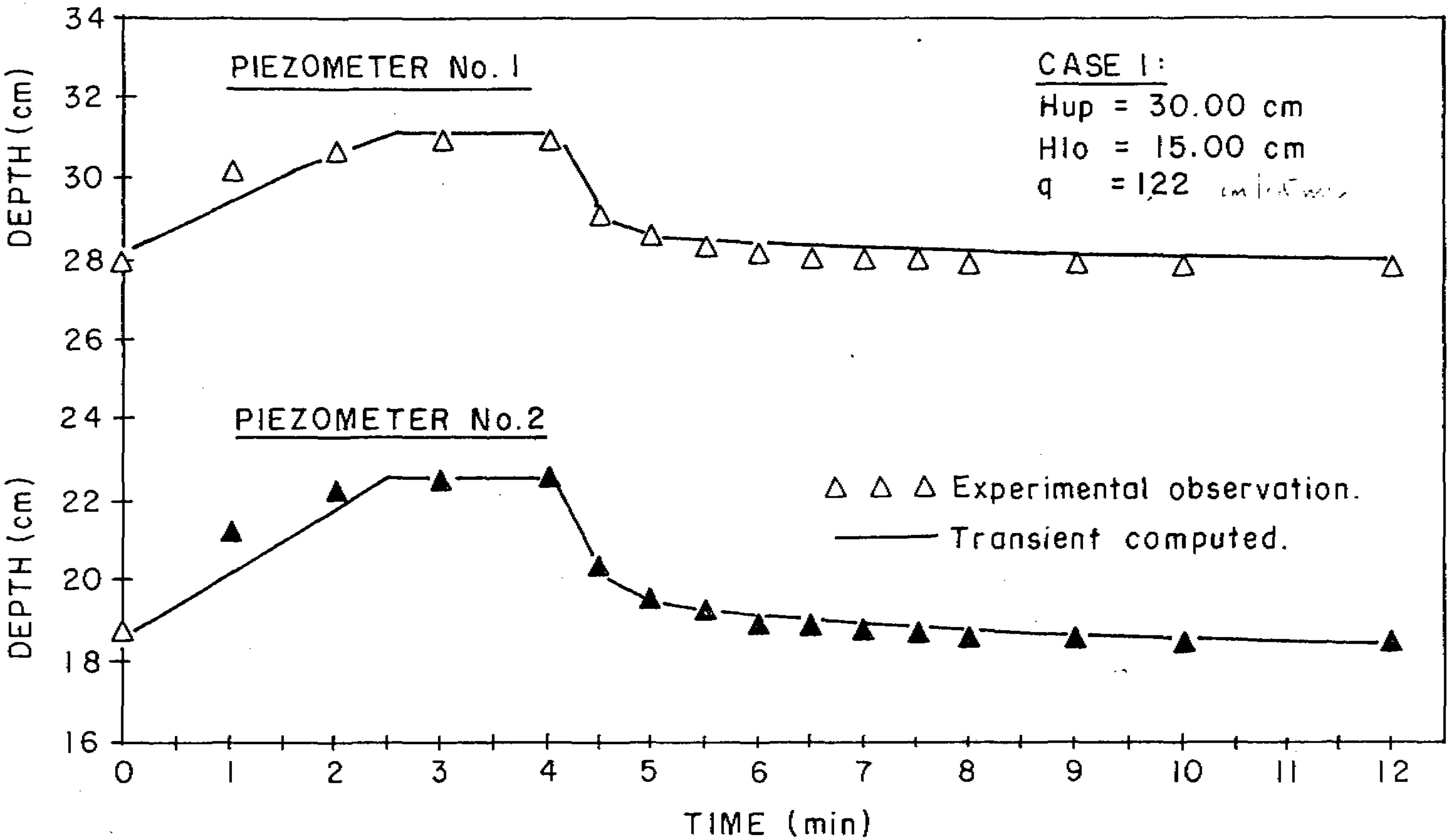


Figure 5 Comparison between transient computed and experimental observation showing the water table elevation in response to recharge from unsaturated zone at piezometer points.

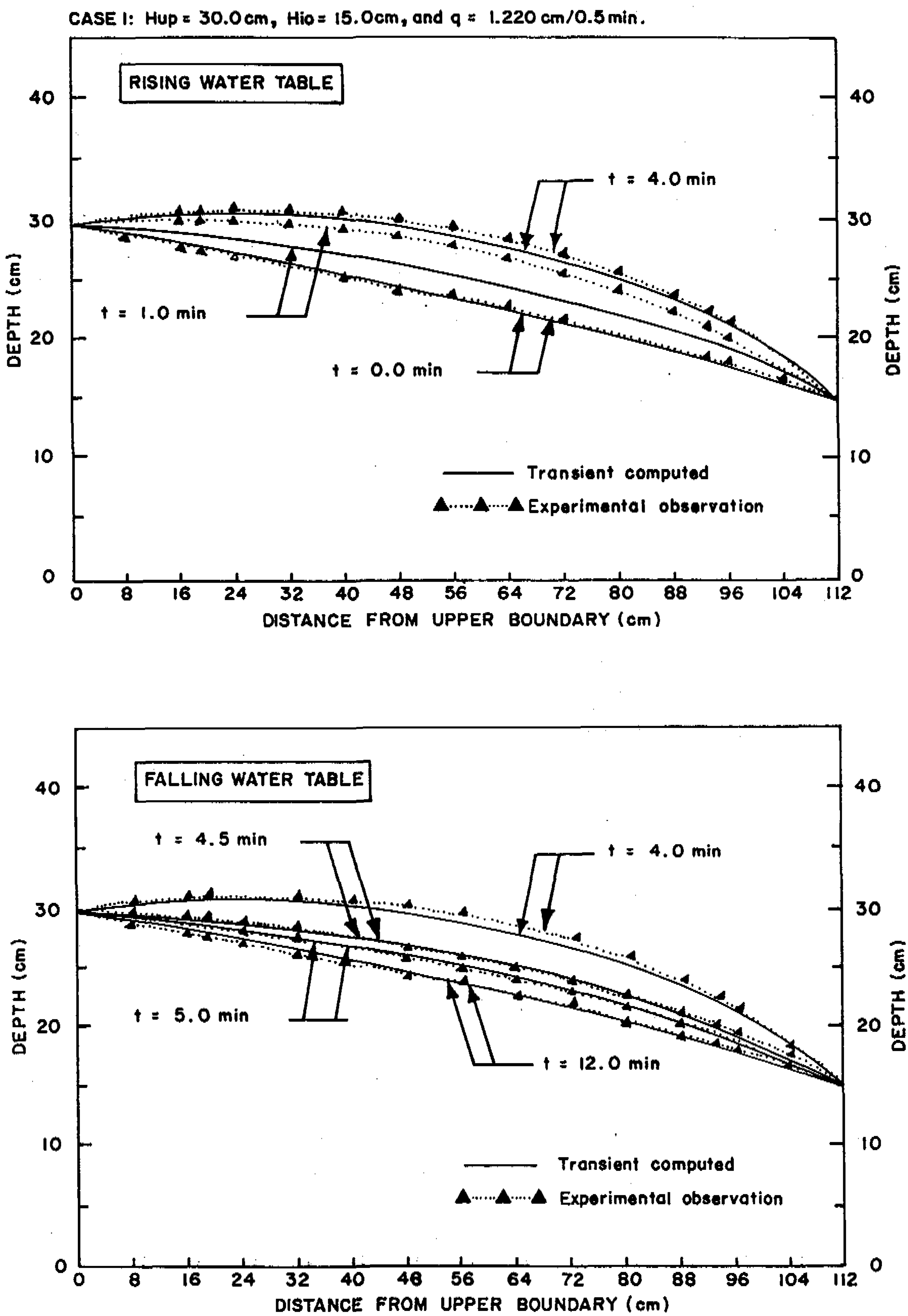
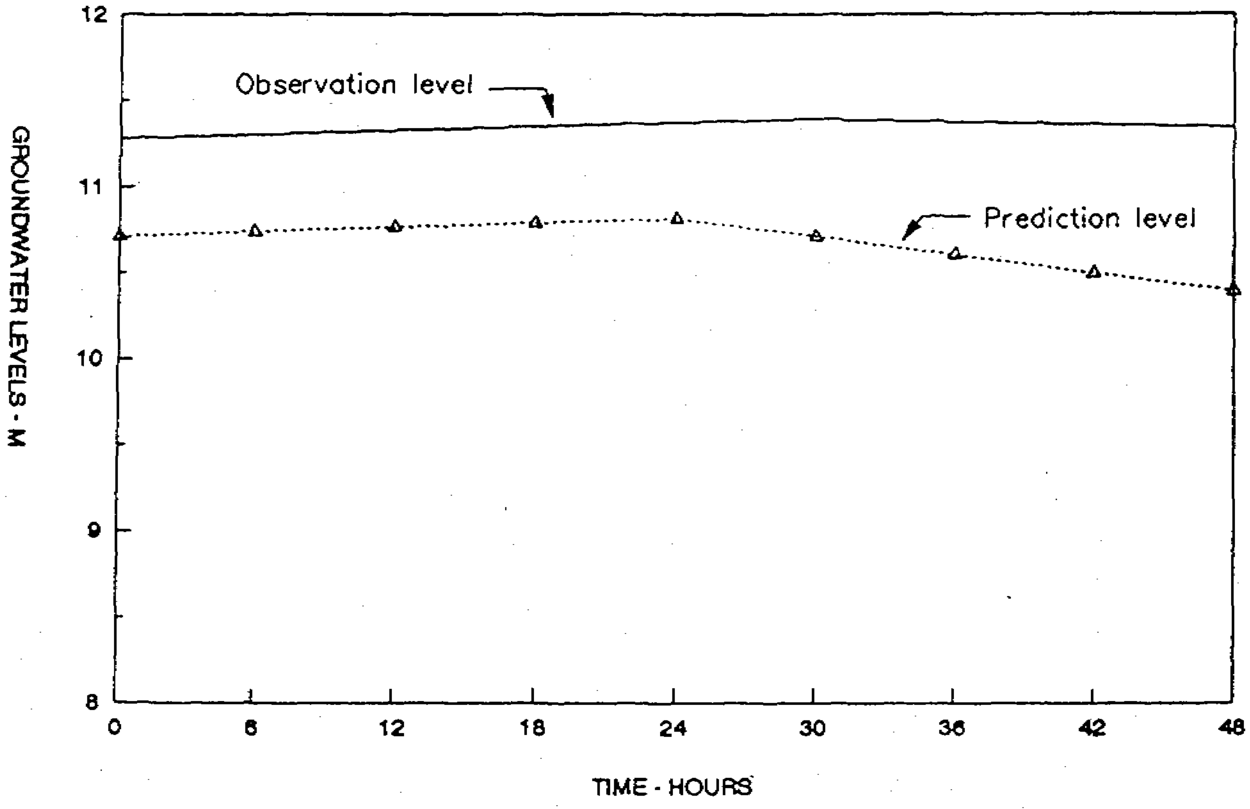


Figure 6 Comparison Between the Transient Computed and Experimental Observation Showing the Free Water Profile in Response to Recharge

Figure 7 Comparison of the groundwater levels prediction between the integrated model and the field observation (BH8)



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OPTIMAL CONJUNCTIVE USE OF SURFACE AND GROUNDWATER RESOURCES IN EGYPT

Manar Z. El-Beshri[1] and John W. Labadie[2]

ABSTRACT

The potential for HAD storage reaching dangerously low levels during drought years has heightened an awareness in Egypt of the need to diversify water development to include withdrawal from shallow aquifers and reuse of drainage water to help alleviate the impacts of drought conditions. A generalized network optimization model for stream-aquifer systems called MODSIM is applied to optimal conjunctive use of water resources in Egypt. MODSIM includes an imbedded groundwater flow model for predicting stream-aquifer interaction and appropriate lagging and attenuation of irrigation return flows and river depletions due to adjacent well pumping. After extensive calibration, a number of management runs were carried out using the critical low flow period of 1980 to 1989. The management runs examine a number of important issues, including: (i) determining the best locations for increased groundwater extraction; (ii) increasing the amount of reusable drainage water as an additional source; (iii) determining optimal HAD release policies under conjunctive use that satisfy all system demands; (iv) determining impacts of reduced optimal HAD releases on irrigation and navigation; and (v) impacts of modifying cropping patterns to reduce irrigation water demands during drought. Results indicate that optimal conjunctive use policies can increase overall water use efficiency from an estimated 73%, under current practice, to 83%, while maintaining higher levels of HAD storage.

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1. INTRODUCTION

Although Egypt is an arid country, water limitations have been further aggravated by drought conditions which the country has been enduring for almost 15 years. Low flows in the Nile River, the main source of water for Egypt, have occurred since 1979. Compensation for water shortage has necessitated increased releases from the High Aswan Dam (HAD) in order to satisfy water demands, which has resulted in severe reductions in storage levels. Further releases from the High Dam under continuing conditions of low flows represents a threat to use of HAD storage for extended drought conditions (Abu-Zeid and Abdel-Dayem, 1990).

Since growing demands in Egypt compete for limited water suppies under existing drought conditions, it is critical that available water resources are managed effectively and efficiently. As low flows continue, it is crucial to maximize efficient usage of existing water resources and consider alternative sources of water to ameliorate water shortage and minimize impacts on crop production, domestic usage and industry. Policies must be developed for releases from the High Aswan Dam in conjunction with contributions from groundwater and reusable drainage water to thwart the severe impacts of continuing low Nile River flows. Egypt must seek other possibilities for providing future water requirements since it has a fixed share of the Nile River flow (Abu-Zeid and Rady, 1991).

On the demand side, the irrigation system in Egypt is known to have an overall efficiency of around 70%. A large amount of drainage water, around 14 milliards m^3 out of an average annual HAD release of 55.5 Milliards m^3, is spilled to the sea. This includes water released for navigation purposes. Reduction or conservation of these losses and optimum management of all available water resources is a vital issue in optimal use of water supplies for Egypt.

The objective is to ensure Egypt's annual water needs for agriculture, municipal, industrial and navigation uses while taking into consideration maintenance of safe levels in the High Aswan Dam. The generalized river basin conjunctive use model MODSIM, developed at Colorado State University (Labadie, 1991), is applied to the main reaches of the Nile river and its primary tributaries in order to allocate available water resources to the required demands. MODSIM calculates the best release policy for the HAD, in conjunction with contributions from groundwater storage and reusable drainage water, for ensuring satisfaction of all demands. Several scenarios representing various potential locations for groundwater extraction and their capacities, including present and future amounts of reusable drainage water, are evaluated. The effect of applying a reduced release policy is also considered, with tradeoffs assessed between the resulting HAD storage at each reduction level and incurred shortages in navigation and other demands.

2. MODSIM RIVER BASIN CONJUNCTIVE USE MODEL

2.1 Basic Assumptions

The underlying principle in the operation of MODSIM is that most physical water resource systems can be simulated as capacitated flow networks. The term *capacitated* refers to the specification of strict upper and lower bounds on all flows in the network. The components of the system are represented in the network as nodes, both storage (i.e., reservoirs and groundwater basins) and non-storage (i.e., river confluences, diversion points, and demand locations) and links or arcs (i.e., canals, pipelines, and natural river reaches) connecting the nodes. In order to consider demands, inflows, and desired reservoir operating rules, several additional *accounting* nodes and linkages are created to insure the fully circulating nature of the network and guarantee satisfaction of flow mass balance throughout the entire system. A fully circulating network requires that all nodes in the network have both inflow and outflow links. More than one link can connect any two nodes. It should be noted that the MODSIM user is only responsible for defining the actual flow network. All *accounting* nodes and links are added automatically by the model.

Important *assumptions* associated with MODSIM are listed as follows:

1. All storage nodes and linkages must be bounded from below and above (i.e., minimum and maximum storage and flows must be given). The latter bounds are allowed to vary over time in the model.
2. Each linkage must be unidirectional with respect to positive flow. Possible flow reversals can be modeled by assigning an additional reverse direction link between two nodes.
3. All inflows, demands, system gains and losses must accumulate at nodes. Increasing the density of nodes in the network thereby increases simulation accuracy, but also increases computer time and data requirements.
4. Import nodes can be designated for water entering the system from across system boundaries.
5. Each reservoir can be designated as a spill node for losses from the system proper. Spills from the system are the most expensive type of water transfer, such that the model always seeks to minimize unnecessary spill. Power spills can be considered through addition of an additional link downstream of a power plant which can be labeled as a high cost link.
6. Reservoir operating policies are provided by the user in the form of target end-of-period storage volumes for each reservoir. Maximum storage capacity can be designated as spill capacity or the bottom of the flood control pool in a reservoir.

2.2 Network Flow Optimization Problem

Within the confines of mass balance throughout the network, MODSIM sequentially solves the following linear optimization problem via the Lagrangian relaxation network flow optimization algorithm over each successive time period in the simulation:

$$\textit{minimize} \sum_{(i,j)\in A} c_{ij} q_{ij} \tag{1}$$

subject to:

$$\sum_{j\in O_i} q_{ij} - \sum_{j\in I_j} q_{ji} = 0; \quad \textit{for all} \;\; i \in N \tag{2}$$

$$\ell_{ij} \le q_{ij} \le u_{ij} \quad \textit{for all} \;\; (i,j) \in A \tag{3}$$

where

A	=	set of all arcs or links in the network
N	=	set of all nodes in the network
O_i	=	set of all nodes with links originating at node i
I_i	=	set of all nodes with links terminating at node i
q_{ij}	=	integer valued flow rate from node i to node j
c_{ij}	=	costs, weighting factors, or priorities per unit of flow rate in the link from node i to node j
ℓ_{ij}	=	lower bound on flow in the link connecting node i to node j
u_{ij}	=	upper bound on flow in the link connecting node i to node j

Equations 2 insure that total flow out of any node equals total inflow to that node, and are referred to as *node constraints*. Equation 3 specifies finite lower and upper bounds on all arc or link flows, and are called *arc constraints*. The terms *arc* and *link* are used synonymously in this development. The notation used in the above equations implies that there is one unique node pair (i.e., beginning and ending nodes) for each arc or link. This is only for notational simplicity, however. Again, the network flow optimization algorithm allows several arcs to share the same node pair. All flows are assumed to be described in volume units per time interval selected for the simulation, and are assumed to be uniformly distributed over the time interval.

Complex river basins can be simulated by appropriate definition of the variable bounds and costs associated with the above network flow optimization problem. The data base for the network optimization problem is completely defined by the *link parameters* for each link (i,j): $[\ell_{ij}, u_{ij}, c_{ij}]$, as well as the sets O_i, I_i, and A. The link parameters are automatically defined by MODSIM, based on data provided by the user.

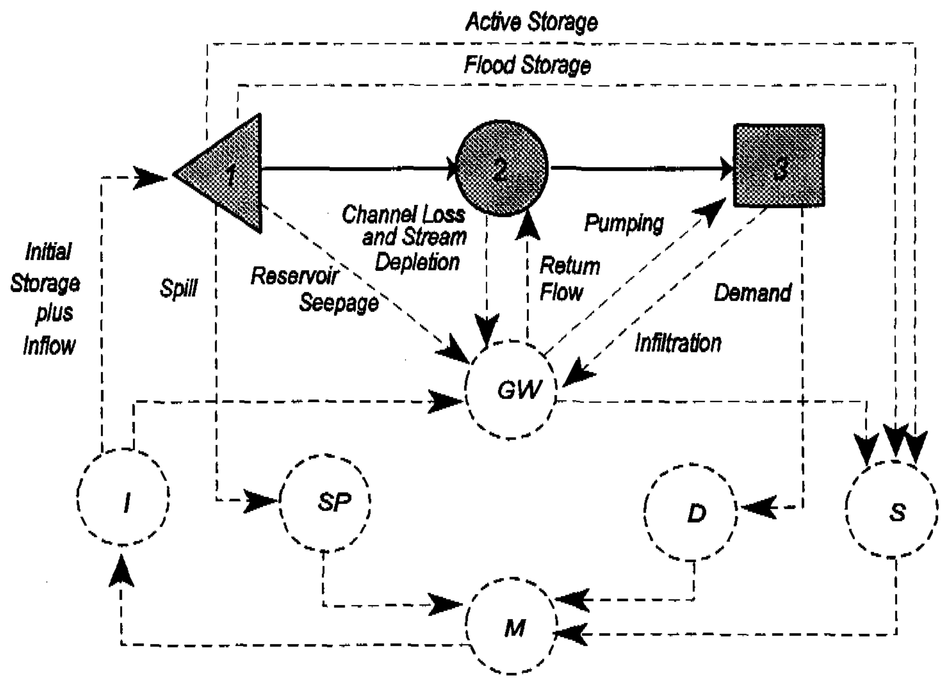


Figure 1. Network structure for MODSIM with accounting nodes and links

An example fully circulating network is shown in Figure 1. Nodes 1, 2, and 3 are actual, physical system nodes. Node 1 is a reservoir, Node 3 is a demand diversion, and Node 2 is an intermediate node. The nodes and links which appear as dashed lines in Figure 1 are representative of special *accounting* nodes and links. That is, they are not part of the physical system, but are included to account for mass balance throughout the entire system. Notice that there are always six *accounting nodes*, but the number of *accounting links* is directly related to the size of the physical network.

The accounting nodes are designated as follows:

I: *accounting inflow node:* collects total system inflows and initial reservoir storage to be distributed to appropriate locations by accounting links

D: *accounting demand node*: accumulates all flows used to meet demands on the system

S: *accounting storage node*: accumulates all end-of-period or carryover storage from reservoirs

SP: *accounting spill node*: accumulates the total volume of spill from storage nodes in the system due to insufficient reservoir capacity; spills are assumed to be uncontrollable and unusable downstream.

M: *accounting mass balance node*: maintains overall mass balance for the network

GW: *accounting groundwater node*: maintains interactions between groundwater and surface water, including return flows and stream depletions due to pumping ; individual groundwater storage nodes may be created by the user.

2.3 Unregulated Inflows

Unregulated inflows may be based on historical data, future forecasts, drought scenarios, or synthetic generation of streamflows. Any real node in the system can be an inflow node. They are connected by accounting links which are directed from the accounting inflow node to each point of inflow. Any node can be designated an inflow node, including a reservoir. Inflows are defined by setting the lower and upper bounds on these accounting links equal to the inflow, thereby guaranteeing that exactly those specified inflows are input. A cost of zero is assigned to these links since these are natural inflows. For accounting links from the accounting inflow node to reservoirs, the links now include any carryover storage from the previous period, in addition to the unregulated inflow.

2.4 Target Reservoir Storage Levels

In addition to inflow links, reservoirs are connected by two additional accounting links for specifying total carryover storage to the next time period. These links originate at each reservoir and accumulate at an accounting carryover storage node. Link [1] is called the accounting active storage link, and link [2] is the accounting flood storage link. The lower bounds on the active storage links are the minimum reservoir storage or dead storage. The upper bounds are user specified end-of-period target storages which represent ideal guidecurve levels for active storage for the current period.

If a large inflow occurs, storage may exceed the target active storage level. Any excess storage is carried in link [2]. Its lower bound is zero (indicating no excess storage above the target level) and its upper limit is the maximum excess space above the target level.

If inflow is so large that spillage must occur, the spills are carried in accounting link [3] and collected at the accounting artificial spill node. Its lower limit is zero and its upper limit is set at total storage capacity in the entire system, multiplied by 10. Spills are assumed to be lost from the water supply system. It is generally a good idea to specify all reservoirs as spill nodes, if possible.

The costs c_{iS} on the accounting active storage links are computed as follows to reflect storage right priorities. For reservoir *i*, the user selects priority $OPRP_i$ as an integer number between 1 and 99. Note that a lower number represents a higher priority. MODSIM then computes a cost c_{iS} as:

$$c_{iS} = -[1000 - OPRP_i \cdot 10] \tag{4}$$

Notice that c_{iS} is a negative number, which represents a benefit associated with carryover storage. The cost associated with flow in the accounting flood storage link is always set at zero. The costs on the accounting spill links are given the highest positive number of any link: 10,000 times the preferential order of spillage.

MODSIM allows the user to input separate target storage levels for each reservoir *i* and for each period *t* throughout the entire simulation. This option is particularly valuable during model calibration and allows input of actual measured storage levels in the system over the historical period in order to compare computed downstream flows with gaged flows. In addition, target storage levels may be input as a function the system hydrologic state for any month. The hydrologic state is defined by storage plus inflows in any user-specified subsystem of reservoirs. Target storage levels may be defined for up to three possible ranges of the hydrologic state, which are defined as *DRY*, *AVERAGE*, and *WET*.

2.5 Evaporation Loss

The program accepts a variable number of elevation-area-capacity data points for any reservoir. Relations between storage and surface area are used to compute evaporation loss as a function of average surface area over the period. Elevations need only be input if hydropower is generated at a storage node. Setting them to zero indicates that there is no hydropower at that node.

Evaporation loss from reservoirs is accounted for as follows. Compute for each reservoir *i*:

$$E_{i\max} = e_i \cdot [A_i(S_i) + A_i(S_{i\max})]/2 \tag{5}$$

$$E_{i\min} = e_i \cdot [A_i(S_i) + A_i(S_{i\min})]/2 \tag{6}$$

$$E_{itarget} = e_i \cdot [A_i(S_i) + A_i(T_i)]/2 \tag{7}$$

where e_i is net evaporation rate for reservoir i (e.g., meters per month) for the current period: A_i is the (interpolated) area-capacity table for reservoir *i*, S_i is storage at the beginning of the current period, S_{imax} is the maximum capacity, S_{imin} is dead storage, and T_i is the user supplied target level.

The storage link parameters are then adjusted as follows:

for active storage links: $[(S_{imin} + E_{imin}), (T_i + E_{itarget}), c_{iS}]$
for flood storage links: $[0, (S_{imax} - T_i) + (E_{imax} - E_{itarget}), 0]$

This means that link upper bounds are adjusted to carry sufficient flow to account for evaporation loss, and the lower bound on the active storage link is increased so that when evaporation is removed, it will not be violated. After calculations for the current period are completed, actual evaporation is computed based on average surface area over the period, and then subtracted from total flows in the carryover storage links. This net flow becomes the carryover storage for the next period.

2.6 Hydropower Generation

Program MODSIM has the capability of computing both power capacity and energy production in a hydroelectric system. The basic power equation used in MODSIM is:

$$P = K \cdot Q \cdot H \cdot e(Q,H) \tag{8}$$

where:

P	=	mean power output in kilowatts
Q	=	reservoir release (volume/period)
H	=	(mean effective head) = (mean gross head on turbines) - (mean tailwater elevation) - (head loss)
e	=	overall plant efficiency, which can be entered as a table of values as a function of discrete Q and H. These tables can include consideration of hydraulic losses and tailwater affects during high flow periods
K	=	3.729 for Q in $10^6 m^3/mo$ and head in m; 16.214 for Q in $10^6 m^3/wk$.

2.7 Flow-Through Demands

Program MODSIM provides for demands for water which are not terminal; i.e., demands which flow through the demand node and remain in the network for subsequent diversion. This would include instream uses for navigation, water pollution control, fish and wildlife maintenance and recreation. Flow-through demands are also useful for augmentation plans, exchanges between basin water users, and development of reservoir release operating rules. In effect, the flow-through demand operates by iteratively removing flow as a demand from the network, but then replacing the flow at a specified (usually downstream) accrual node. The flow is replaced by adding it as an inflow to that node via the accounting arc connected to the accounting initial storage and inflow node.

The use of a flow-through demand for minimum streamflow requirements has two primary advantages: (i) the flow-through demand can be assigned a priority just like any other demand in the basin, and (ii) simply setting a fixed lower bound on the link corresponding to a minimum streamflow requirement can result in an infeasible solution if there is insufficient flow available to meet the demand. The flow-through demand can receive a shortage just like any other demand, depending on the relative ranking of the water right priority.

2.8 Consumptive Demands

The other type of demand allowed in MODSIM is when all water contributing to demand satisfaction in the model is assumed to be consumptively used in the system. These are termed terminal or consumptive demands. However, the ability of MODSIM to consider stream-aquifer interactions allows calculation of subsurface return flows to the river due to inefficient application of water. The demands ultimately dictate the amount of water released upstream, rather than attempting to determine a priori release rules. Note that a storage node can also serve as a demand node, as well as an inflow node.

Demands can be input as decreed water right amounts, historical diversions, predicted agricultural demands based on evapotranspiration calculations (performed outside the model), and projected municipal and industrial demands.

An accounting demand arc connects each real node (storage and non-storage) with the accounting demand node. The lower bound on this arc is set equal to zero, while the upper bound is set equal to the demand associated with each real node. The cost placed on each accounting demand arc is calculated by the following equation:

$$c_{iD} = -[1000 - (DEMR_i \cdot 10)] \tag{9}$$

where:

c_{iD}	=	negative cost per unit flow indicating priority for satisfying demand at node i
$DEMR_i$	=	user input priority for meeting demand at node i (number between 1 and 99, with a lower number representing a higher priority)
D	=	accounting demand node
i	=	demand node

2.9 Links and Conveyances

All physical links in the network must be bounded from above and below. MODSIM includes the capability of allowing the user to input a constant bound for each link, or varying daily, weekly or monthly maximum flow limits for some links. The latter are useful for considering seasonal influences in canal capacities and maintenance schedules.

For certain problems where it would be desirable to include pumping costs, MODSIM provides the additional option of user input of costs for any linkage in the network. Negative costs can be entered to represent benefits, such as from low head hydropower production. Costs (positive or negative) can be assigned to any link by the user to discourage or encourage, respectively, flow in that particular link according to predefined operational criteria.

MODSIM includes the capability of removing channel losses directly. A loss coefficient for each reach is included in the data input. This coefficient represents the fraction of flow at the initial node of the link that is lost during transition through the link. An iterative procedure is employed in MODSIM for calculating channel losses, and proceeds as follows: First, network flows are solved via the

Lagrangian relaxation algorithm with no losses assumed. Initially, all flows are set to zero, or the lower bound is greater than zero. The losses in each link are computed by multiplying the loss coefficient by the calculated flows, and this loss is removed at the node at the beginning of the link and sent to the accounting groundwater storage node, which is automatically set up by MODSIM. The Lagrangian relaxation algorithm is then solved again with the network flows so modified. New link losses are then computed and the procedure is repeated until acceptable convergence has occurred.

2.10 Stream-Aquifer Interaction

The stream-aquifer module within MODSIM allows the user to consider reservoir seepage, irrigation infiltration, pumping, channel losses, return flows, river depletion flows due to pumping, and aquifer storage. Stream-aquifer return/depletion flows are simulated using response coefficients calculated using the one dimensional equations developed by Glover (1974), McWhorter (1972), and Maasland (1959). Groundwater response coefficients estimated from other methods such as the SDF method, the 3-D groundwater finite difference model MODFLOW/MODRSP (Maddock and Lacher, 1991), or the discrete kernel generator, GENSAM (Morel-Seytoux and Restrepo, 1987) can be read as external data files. If spatially distributed stream-aquifer response coefficients have been generated using MODRSP, they can be used to allocate groundwater return/depletion flows to multiple return/depletion flow node locations anywhere in the river basin network system.

Figure 2 illustrates the mechanisms for stream-aquifer interaction utilized in MODSIM. Again, the groundwater response coefficients $COEF_i$ for lagging groundwater return flows and depletions from pumping can be calculated internally based on the equations of Glover (1974), or calculated externally using finite difference groundwater models such as MODFLOW. For internal calculation, the user must input lumped aquifer parameters (storage coefficients and transmissivities), as well as average distance of the irrigation area, canal, or well from the river.

3. NETWORK STRUCTURE FOR LOWER NILE RIVER BASIN

3.1 Network Configuration

The physical features of the system under study have been placed in network structure of 60 nodes and 61 links. The High Aswan Dam, all head regulators, all river reaches and first order canals are represented by links and nodes as shown in Figure 3. These are actual screen displays of the network structure using the graphical user interface (GUI) for MODSIM.

Notice in Figure 3 that additional links have been added from terminal nodes to allow for spillage. These links are given a high penalty so that spill will not take place unless it is necessary. The final nodes spill excess flow to the sea. A terminal spill node No. 61 has been added to accumulate total spill from both Nile branches: Damietta branch to the east and Rosetta branch to the west. This node represents the total spill from the system that goes to the sea. It is given large demand, but at an extremely low priority so that the model minimizes the amount of spilling.

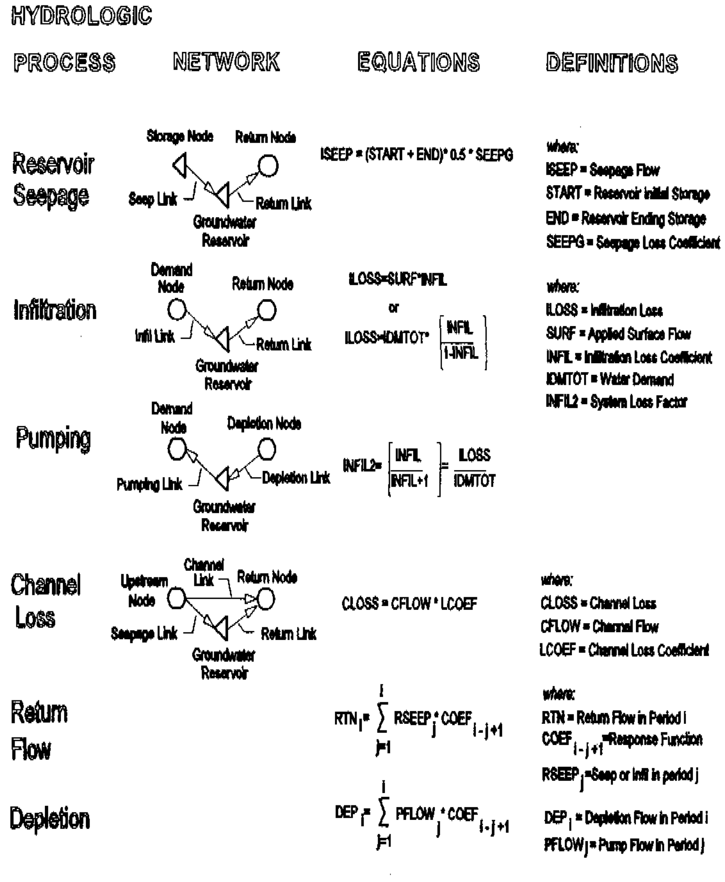
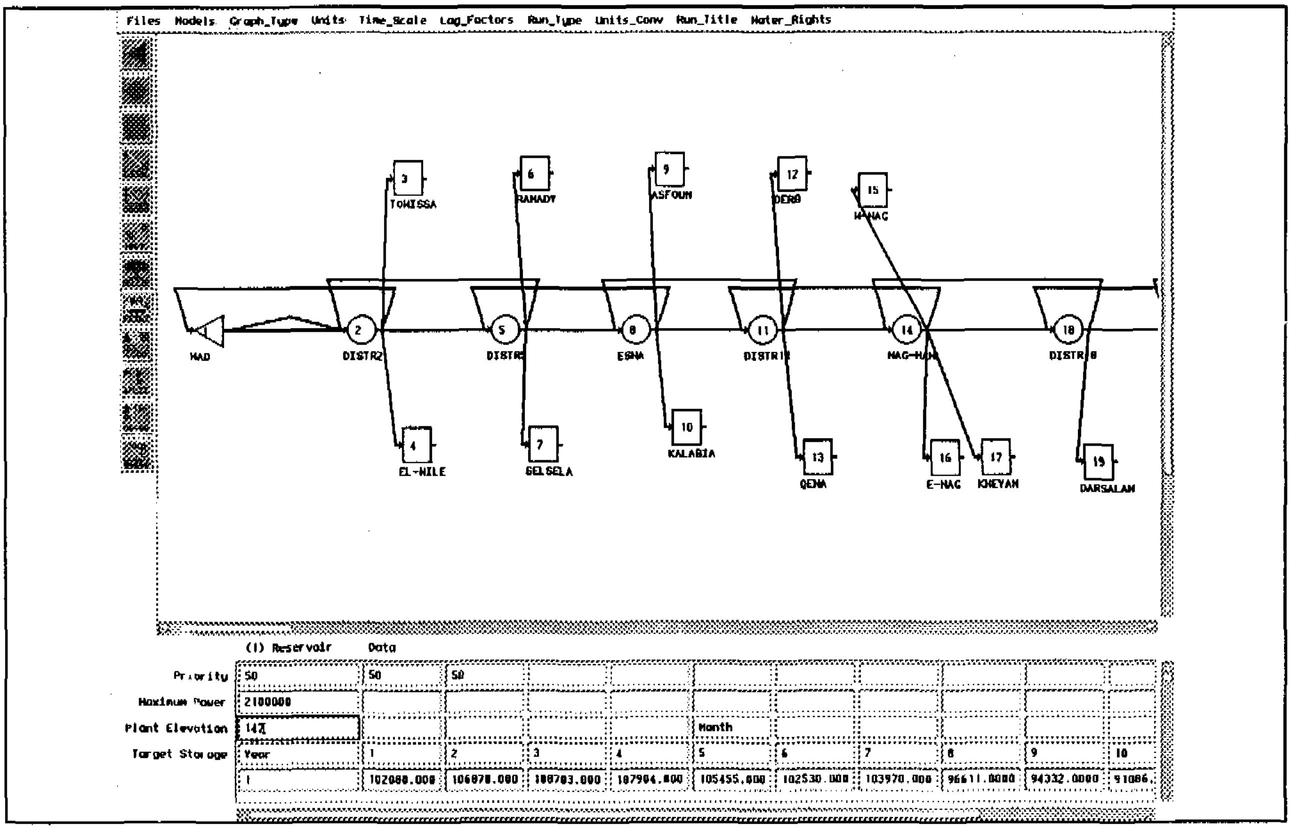


Figure 2. MODSIM Stream-Aquifer Functions

3.2 Navigation Links

Other links have been added to the main reaches in an opposite direction to the reach flow. The addition of these links does not mean that flow is actually moving upstream. These links are assigned a maximum bound equal to the minimum navigation flow for the link representing the actual direction of flow. These reverse links are given a penalty cost to ensure that the net downstream flow in that reach will remain above the minimum navigation flow, unless there are shortages in the water supply. All main Nile reaches, from the High Aswan Dam to Assiut Barrage have a minimum flow bound of 2490 Million m^3/month to ensure meeting navigation requirements. From Assiut Barrage to the Delta Barrages, the minimum flows are 2100 Million m^3/month. No navigation requirements are specified for the Damietta branch, whereas the Rosetta branch requires additional releases during the low demand period from December to February for navigation.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HAD
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DISTR2
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ESNA
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14
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EL-NILE
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GELSELA
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KALABIA
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QENA
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E-NAG
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KHEYAN
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Target Storage Year 1 2 3 4 5 6 7 8 9 10
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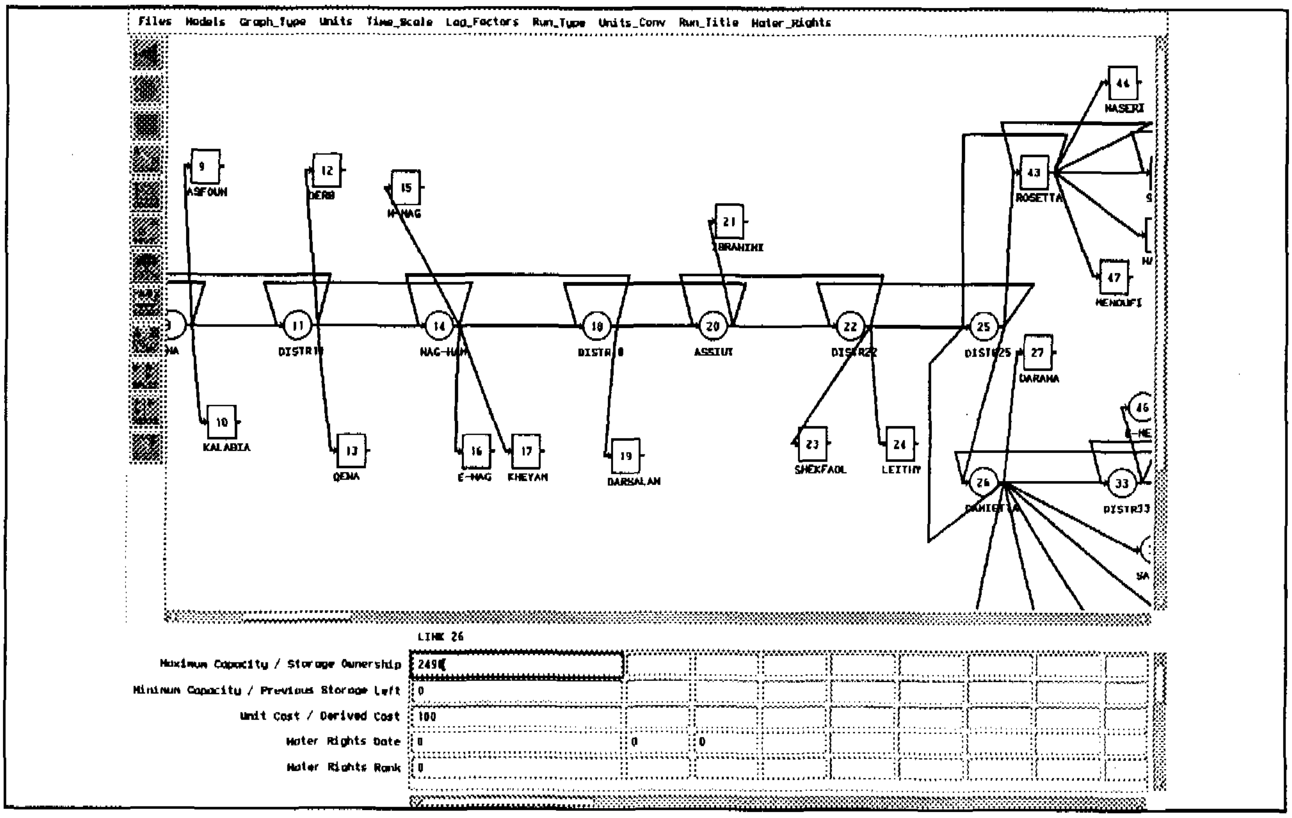
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SHEKFADL
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LEITHY
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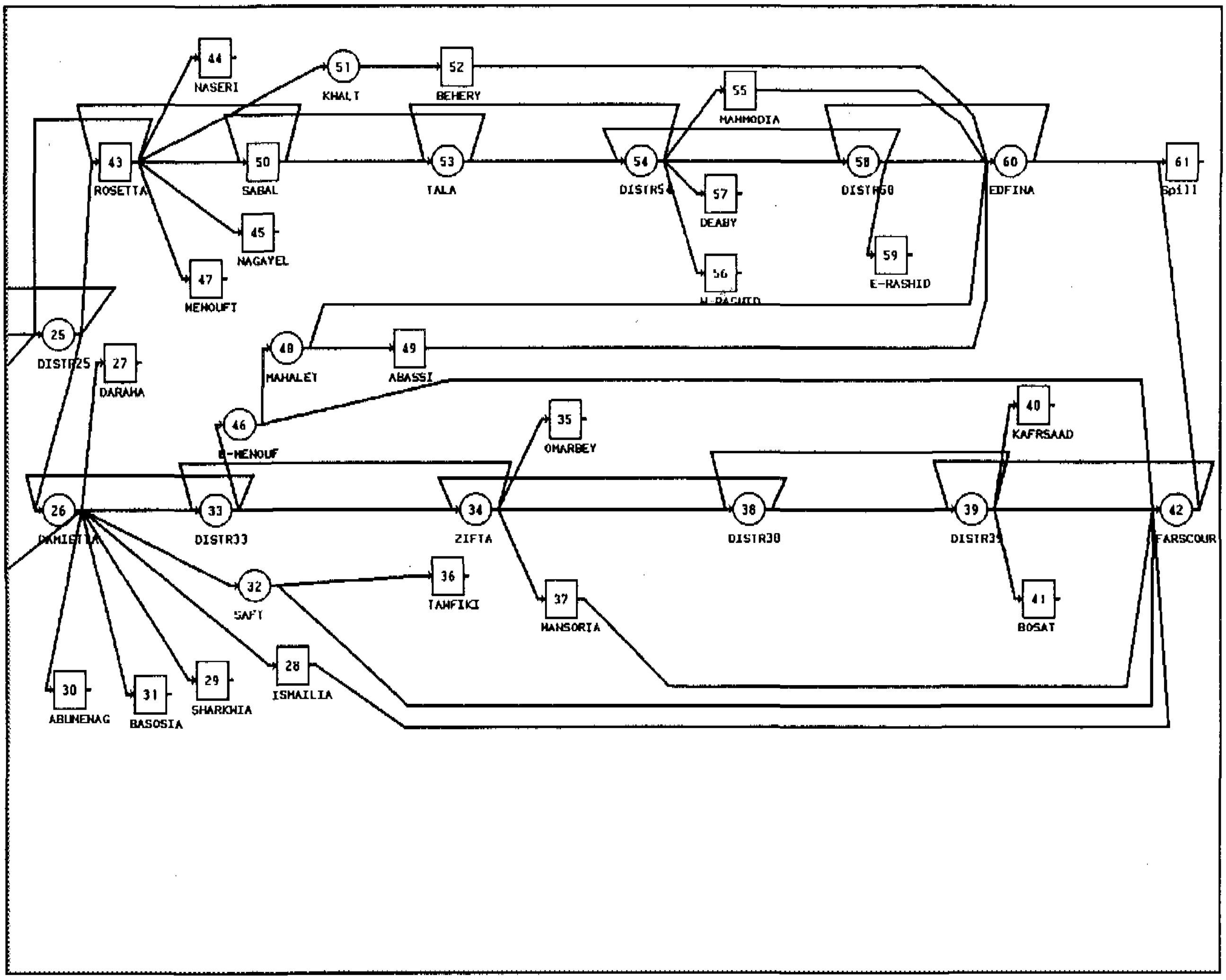

Figure 3. MODSIM Graphical User Interface Display for Lower Nile River Basin

3.3 High Aswan Dam

The High Aswan Dam node (Node 1) is set with a maximum storage capacity of 168.9 Milliards m^3 and a minimum storage capacity of 31.6 Milliards m^3. The reservoir geometry is represented by 18 area-capacity-head points. Evaporation from the HAD reservoir varies according to the average lake surface area over the period, which in turn depends on initial and end-of-period storage in the reservoir. The annual evaporation rate is estimated at 2.7 meters per year, which is then distributed into monthly rates using monthly distribution coefficients.

The Ministry of Public Works and Water Resources does not maintain measurements on seepage rates from the HAD. Average annual seepage loss has been estimated at approximately two Milliards m^3. Bank storage is known to follow a seasonal pattern of flows that seep into and out of the reservoir according to reservoir levels; however, there are no precise data available to account for this term. MODSIM is used during the calibration run to provide an estimate for the seepage rate based on known storage levels and releases during the calibration period.

For hydropower generation, a negative cost is assigned to the hydropower generation link originating from the HAD. Since a negative cost acts as a benefit in a cost minimization model, this ensures that all discharges from the HAD first pass through the turbines for power generation, up to the available discharge capacity. This link has a maximum bound of 9020 Million m^3/month, representing the maximum capacity of the turbines, and a minimum of 2490 Million m^3/month to ensure satisfaction of downstream navigation in the main Nile channel during months of low irrigation requirements.

3.4 Upper Nile Inflows

Inflows to the Nile River system include inflows to the HAD, reusable drainage water pump station flows, and irrigation return flows. Natural inflows to the HAD reservoir come from the Upper Nile Basin, out of which Sudan has a share of 12.7 Milliards m^3. Gabal Awleya abstractions amount to 2.108 Milliards m^3, which should be subtracted from the natural inflows to calculate actual inflows to the HAD in Egypt. Sudan and Gabal Awleya abstractions follow a specified monthly distribution.

3.5 Reusable Drainage Water

Reusable drainage water pump stations are represented by inflows to nodes according to their locations with respect to the main reaches and canals, and are only used for management runs. For the calibration run, the demands estimated for the historical period already include reusable drainage water. Locations of nodes for reusable inflow have been approximately determined according to the location of feedpoints of the pump stations with respect to the irrigation network. Unpublished data from the Water Master Plan reports for the year 1988-1989 give the names, locations and amounts of reusable drainage water at each reuse pump station. The total amount of reusable drainage water sums to 2.67 Milliards m^3 for the year 1988, and is distributed over the year based on specified monthly coefficients.

3.6 Irrigation Return Flows

Irrigation return flows are calculated as 35% of the flows drained from certain areas, which eventually return as available inflows to the system. In Upper Egypt, return flows reach the main Nile reach via spillways. They are considered as gains to the system that are usually high during the months of low releases (i.e., September through March), and low during the high release months of April through August. They usually occur between the Assiut and Delta barrages and serve to satisfy some of the downstream demands. In the Delta area, however, a portion of the return flows are reused via pump stations that feed some of the canals. The remainder of the return flows unsuitable for reuse are spilled the sea.

Return flows are represented by inflows to nodes (5, 8, 20). These return flows are assumed to be equal to 30%-35% of the amount of flow released to the canal network. These flows occur in some of the main Niles reaches via spillways in Upper Egypt, and are considered as another external source of inflow. They originate from bankstorage and groundwater, which makes their occurrence mostly during months of low flow in the system. The values for these return flows are calculated by the Ministry of Public Works and Water Resources between each two barrages located on the main Nile reach. As for Lower Egypt, better known as the Delta area, only a small part of return flows are reused for irrigation via pump stations that discharge flows to some of the canals or directly to the Rosetta and Damietta Branches. The remainder of the return flows not found suitable for reuse are spilled to the Sea.

3.7 Demands

Irrigation demands were calculated according to data obtained from the Ministry of Public Works and Water Resources. The Ministry is known to adopt the policy of fulfilling irrigation demands as a first priority by ensuring that monthly releases are equal to the calculated demands according to cropping pattern, municipal and industrial demands, conveyance losses, and navigation demands. However, releases obtained from the Ministry of Public Works and Water Resources do not coincide with calculated demands based on the given cropping pattern for the same year, along with estimates for municipal and industrial demands and navigation requirements. Reusable drainage water and return flows are also taken into consideration in these calculations.

Municipal and industrial demands were obtained from Water Master Plan reports for the year 1972 for each canal. The percentage of increase in the total municipal demands from the year 1972 to 1988 was calculated knowing the total demand for municipal purposes in 1972 and in 1988. The percentage increase in total industrial demands from the year 1972 to 1988 were also found. The calculated percentages of increase in the municipal and industrial demands were used as factors to obtain recent municipal and industrial demands for each canal. Calculated municipal and industrial demands were then added to the irrigation demands.

3.8 System Losses

Losses form a significant portion of the system releases. Application losses are accounted for in the water requirements for each crop, representing the difference between the amount of irrigation water supplied to the system for adequate irrigation, less the evapotranspiration by crops. Application losses are generally estimated at 15% in Egypt. Conveyance losses are divided into losses occuring between the HAD release point and the canal intake, and the portion lost within the canal itself. The overall loss rates are estimated by the MPWWR as 10% for Upper Egypt, 15% for Middle Egypt and 20% for Lower Egypt. These losses include seepage loss, evaporation loss, and losses due to the weed growth in the channels. They are included by adding them on to the demands calculated for each canal.

The overall efficiency of the system is an aggregated loss term that combines both the application and conveyance losses. It serves to evaluate the overall performance of the system, which is estimated at 70% by MPWWR. The calibration run is used to determine the overall efficiency of the system.

3.9 Groundwater Modeling

MODSIM is used to calculate stream-aquifer interactions from groundwater pumping and return flows internally. Since data were not readily available data on lengths of the various canals in the system, the center of gravity for each area served was estimated by dividing the area by the width of that same area to obtain the length. This value was then divided by two to obtain an approximate estimate of distance from the river to the the center of gravity of that area. Transmissivity values are obtained from the Water Science Magazine (1991) published by Water Research Center. Transmissivity increases gradually from the HAD to Nag Hamadi barrage then decreases until Cairo. It then increases from Cairo towards the Delta area. The specific yield is estimated to equal 0.15 (Water Science Magazine, 1991).

Points of depletion in the mainstream and points where return flows accrue are selected in a way that ensures that the pumping node, the depletion node, and the return flow node are interconnected. In case of modeling groundwater in the Delta area, return flows calculated by the model replace the inflows representing reusable drainage water at each pumpstation. For Upper Egypt, all return flows calculated by the model are assumed to be reusable for irrigation.

4. CONJUNCTIVE USE MODELING RESULTS

4.1 Model Calibration

The MODSIM conjunctive use network flow model was first calibrated for the year 1988-1989, for which a complete set of data were available. The purpose of the calibration run was to: (i) simulate the system during that year and compare with recorded flows, (ii) estimate seepage rates from the High Aswan Dam; (iii) establish the best priority configuration for demands in relation to HAD storage, and (iv) find the overall efficiency of the system.

The seepage rate from the reservoir was considered as a calibration parameter for the HAD. MODSIM was set to find the seepage rate as a fraction of average reservoir storage over the period that would result in the correct mass balance for the reservoir. HAD storage levels were obtained from the HAD storage-level rating curve, based on known actual monthly pool levels for 1988-1989. Actual releases from the reservoir for that year were established as outflows from the HAD. This was accomplished by enforcing a flow-through demand equal to the actual releases observed for that year at *dummy* node no. 62 inserted downstream of the HAD. The flow-through demand accrues to the following node on the reach, which is node 2. This approach ensures that releases from the dam are equal to the actual releases for that year.

The system demands used for the calibration run were calculated by the Ministry of Public Works and Water Resources for each main canal. These demands include irrigation requirements, municipal and industrial requirements, minus reused drainage water as another source of inflow; plus application and conveyance losses. During months of low demand (i.e., December through February), releases were mainly set to meet navigation requirements.

Minimum navigation releases for each month were formulated by assigning a minimum flow of 2490 Million (m^3/month) along the main Nile reach from the HAD, node no.1, to Assiut barrage, node no.20, and a minimum flow of 2100 Million (m^3/month) from Assiut Barrage, node no.20 to Delta barrages, node no.25. A minimum flow of 357 Million (m^3) in December, 1324 Million (m^3) in January, 610 Million (m^3) in February, should be maintained downstream of the Delta barrages on the Rosetta Branch for navigation purposes. These values were obtained from available historical records. A flow-through demand has been set from the Delta barrages on the Rosetta Branch (node 43) to the next node on the reach (node 50) to ensure matching those flows during these 3 months.

Return flows were modeled as inflows to the system in Upper Egypt according to gains tables obtained from the CIDA report, 1990. These return flows attain their highest values during the months of low flow (i.e., October through March) and diminish during months of high flow. Seepage to the channel, bank storage and groundwater sources all contribute to these return flows.

As for Lower Egypt, or the Delta region, return flows are calculated at 21% since only a portion of the 28% losses are available as return flows. Of the releases made to the network, only a small part of the return flows are reused for irrigation via pump stations that discharge to some of the canals or directly to the Rosetta and Damietta Branch.

Calibration runs indicated that the best reservoir mass balance is achieved at a seepage rate of 0.002, which totals to an amount of seepage loss of two Milliards m^3 from the reservoir. Figure 4 shows the actual HAD storages versus the simulated storages in that case. The model simulated storages were higher than the actual storage from September to May, but were perfectly matched for the remainder of the year. Additional storage in the reservoir above the actual storage could be due to following the HAD level-storage rating curves, which have not been updated for many years. Consequently, accumulated sediments in the reservoir may require updating of the original rating curves. Moreover, there are certain physical processes such as bank storage and leakage from the dam that are not accounted for in MODSIM. All these factors have contributed to simulating the reservoir storage slightly deviating from the actual values.

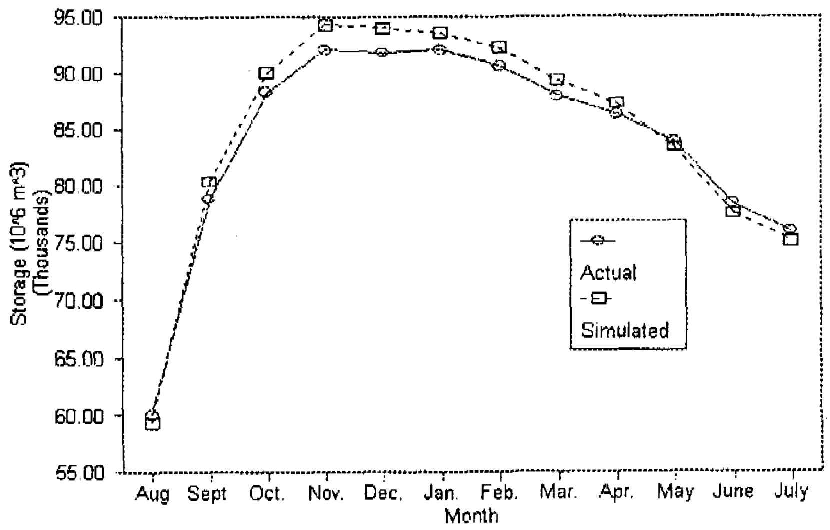

Figure 4. Actual HAD Storages versus Simulated Storages at Seepage Rate = 0.002

The next step is to simulate water allocations to the system by matching the simulated and observed flows through certain checkpoints along the system, and to calculate the overall system efficiency. Using a seepage rate of 0.002 as a fraction of the HAD storage, and giving the HAD storage a slightly lower priority than the HAD releases, a calibration run was made to simulate the historical water allocations to the system, the amount of water spilled to the sea and to determine the system efficiency.

In this run, downstream releases from the Damietta and Rosetta branches were forced to equal the historical releases. Checkpoints at which historical releases were compared with MODSIM releases include: (i) downstream of High Aswan Dam; (ii) upstream and downstream of Damietta Branch Delta Barrages; (iii) upstream and downstream of Rosetta Branch Delta Barrages; and (iv) spills from the system to the Mediterranean.

The calibration run resulted in reasonably good correspondence with historical data. Figure 5 compares actual versus simulated releases upstream of the Delta barrages at the Damietta Branch. This comparison provides the largest deviation in comparison with all the other check points. The pattern of flows is reproduced well, although MODSIM consistently overestimates the flows. Further refinement in return flow calculations is required to achieve an improved correspondence.

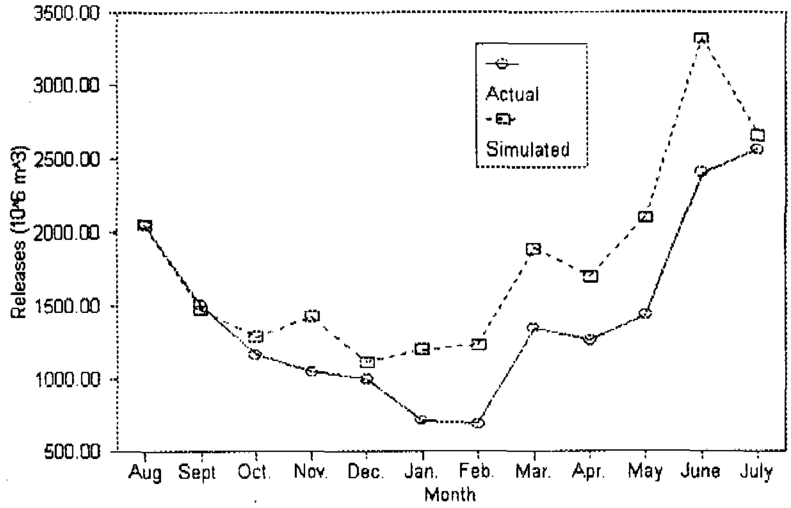

Figure 5. Actual versus Simulated Releases U/S Delta Barrages at Damietta
Seepage Rate From HAD = 0.002

All demands in the calibration run were satisfied, with the exception of slight shortages upstream of the Rosetta Branch Delta Barrages during September. Since releases downstream of that branch were forced to equal the historical records, the model has satisfied this constraint and introduced shortages to the demands located upstream of the Rosetta Branch Delta Barrages. The minimum flows for navigation along the main Nile reach in Upper Egypt satisfied as well. The amount of water spilled to the sea reached 14,400 Million m^3 for the year 1988-1989, as compared with actual estimates of 13,400 Million m^3 for that year. The actual amount of water used by the system found by MODSIM is 39 Milliard m^3 for the year 1988-1989, whereas actual records calculated that value at 40 Milliards m^3 for the same year. The overall efficiency calculated from the model results is at 0.73, whereas the overall efficiency calculated from actual records is estimated at 0.7.

4.2 Conjunctive Use Management Analyses

A critical period of low inflows from 1980 to 1989 is utilized for the management runs. This period began with higher than normal inflows (65610 Million m^3 for the year 1980-1989), and ended with high inflows (99483 Million m^3 for the year 1988-1989).

The initial storage volume at the beginning of the critical period was 103205 Million m^3, which reduced to a critically low level of 41048 Million m^3 in the year 1987-1988. Fortunately, reservoir storage levels increased in the year 1988-1989 to a volume of 75920 Million m^3 due to high inflows for that year. Figure 6 shows actual reservoir volumes obtained from the level-storage rating curves- versus the actual releases versus the historic inflows for the HAD along the nine years of critical period.

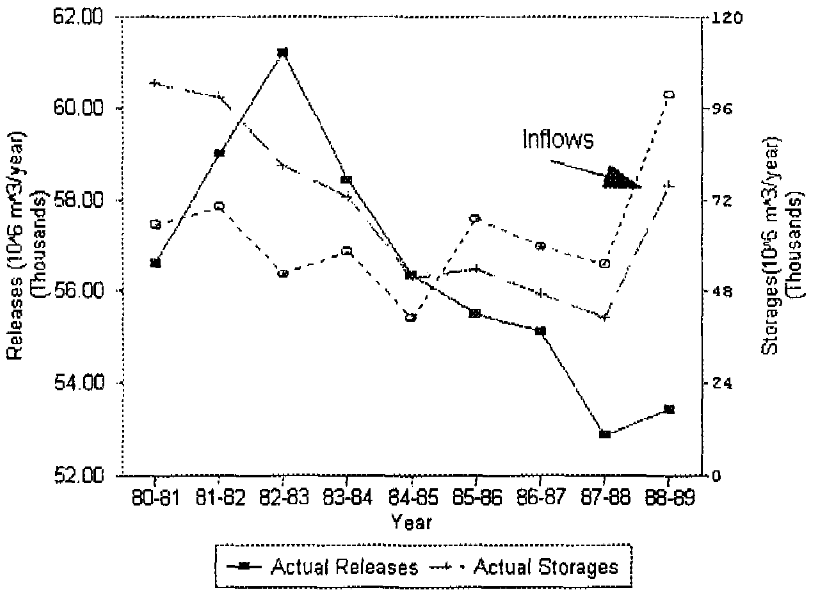

Figure 6. Actual HAD Releases Storages Corresponding to HAD Inflows during the Critical Period of Low Flows

In all management runs, carryover storage in the HAD is given a low priority in relation to downstream canal demands. Navigation is maintained by assigning a cost of -1 to all navigable reaches (which actually represents a benefit), and a high penalty to the flow in reverse links, which penalizes navigation shortages.

A number of possible conjunctive use policies were evaluated using the MODSIM model. These include:

1. ***Using Surface Water for all Demands Except for Municipal and Industrial Demands at Cairo:*** This scenario was introduced to find the minimum HAD releases that can be reached when using groundwater pumping for municipal and industrial requirements at Cairo only. The results of this run show that including groundwater as another source of water for Cairo can reduce releases from the HAD, but only slightly.

2. ***Using Groundwater for Certain Demands in Upper Egypt:*** In this scenario, more groundwater pumping occurs in Upper Egypt, along with present values for reusable drainage water in the Delta area. Pumping at Kalabia canal (node 10), Derb P.S. (node 12), West Nag Hamadi Canal (node 15), Kheyam P.S. (node 17), Ibrahimia Canal (node 21), Sheikh Fadl P.S. (node 23), and municipal and industrial demands at Cairo (node 24). The amount of groundwater pumping in this scenario reaches 7149 Million m^3/year in Upper Egypt, which means reduced releases from the HAD are needed to meet the demands, and more storage is retained than in the previous scenario.

3. ***Using Groundwater for CertainDemands in Upper and Lower Egypt:*** In addition to the pumping allowed in Upper Egypt in the previous scenario, additional groundwater pumping is allowed in Lower Egypt (i.e., the Delta region) at Tawfiki Rayah (node 36), Menoufi Rayah (node 47),

and Behery Rayah (node 52). The pumping capacity assigned to Lower Egypt is equal to the pumping capacity assigned to Upper Egypt in the previous scenario. The amount of groundwater pumping calculated by the model reaches 12817 Million m^3/year in Upper and Lower Egypt. It is observed, however, that less release is required from the HAD when groundwater is extracted from Upper Egypt only, with only reusable drainage water applied in Lower Egypt without additional groundwater pumping. Since required pumping capacity has doubled in this case, there appears to be little benefit of this scenario over previous cases.

4. ***Adding more Groundwater in Upper Egypt with Present Reuse of Drainage Water in Lower Egypt:*** In this scenario groundwater pumping is increased in Upper Egypt, along with reusable drainage water in Lower Egypt, without additional pumping. Groundwater pumping in Upper Egypt is now added to Towissa canal (node 3), Asfoun canal (node 9), Qena P.S. (node 13), East Nag Hamadi (node 16), in addition to nodes designated for pumping in the previous scenarios. The amount of groundwater pumping calculated by the model reaches 13112 Million m^3/year in Upper Egypt under this scenario. Spills to the sea are calculated at 9316 Million m^3, as compared actual spill estimates ranging from 14000 Million m^3 to 16000 Million m^3. It is concluded that increasing groundwater pumping in Upper Egypt has a positive impact on HAD releases and storage, in conjunction with use of reusable drainage water in Lower Egypt without additional groundwater pumping.

5. ***Maximum Groundwater Use in Upper Egypt with Future Planned Reuse of Drainage Water in Lower Egypt:*** In this scenario, groundwater pumping is increased in Upper Egypt to a maximum extent, along with assumptions of future planned levels of reusable drainage water rather than present levels. In addition to Upper Egypt pumping represented in the previous scenarios, pumping is added to El-Nile P.S. (node 4), Ramady Canal (node 6), Selsela canal (node 7), and Dar El Salam P.S. (node 19). Total releases from the HAD over the nine years sum up to 445,000 Million m^3 , as compared with actual total releases of 508,513 Million m^3. Figure 7 shows actual reservoir releases over the nine years compared with the minimum releases that could have taken place with use of additional groundwater in Upper Egypt and present reusable drainage water in Lower Egypt, versus maximum groundwater pumping in Upper Egypt along with future reusable drainage water in Lower Egypt that would have served to satisfy all demands. The amount of water spilled to the sea reduces to 8484 Million m^3 under this scenario. This represents a savings of more than 3177 Million m^3 as a result of increasing groundwater pumping in Upper Egypt.

Applying the optimal release policy increases the efficiency of the system from 73% to 83%, due to increased exploitation of the water that infiltrates the soil, and heretofore considered as losses, which recharges the aquifer and enhances return flows for possible reuse. Another cause for the improvement in system efficiency is the increase in reusable drainage water, which reduces the amount of drainage water spilled to sea.

4.3 Impacts of Further Reductions in HAD Releases

Results have shown that average annual releases at the HAD of 50000 Million m^3 are sufficient to maintain all downstream demands under the optimum conjunctive use policy. The next step is to study possible impacts on irrigation and navigation demands when applying further reductions in HAD releases beyond the optimal conjunctive use policy, as might be necessitated under extreme drought conditions. Table 1 shows impacts of incremental reductions in annual HAD release on storage, irrigation, and navigation. Notice that decreases in HAD releases have a greater impact on navigation demands than

Table 1. Reduction in HAD releases and corresponding HAD storage, spills to Sea and percent shortages

HAD Releases (Mm^3/yr)	HAD Storage (Mm^3)	Spill to Sea (Mm^3/year)	% Irrigation Shortages	% Navigation Shortages
50000	96227	8484	0	0
49000	101707	8345	1	12
48000	107206	8292	2	29
47000	112694	7402	3	34
46000	118189	7148	4	35
45000	123697	6953	6	37
40000	138081	5892	12	50

on irrigation demands. A percentage of shortage of 37% each month is reached in navigation demands when releases are reduced to 45000 Million m^3, whereas only a shortage of 6% is observed at the same release level. The amount of water storage in the reservoir has increases by 27470 Million m^3, but spills to the sea are reduced by 13779 Million m^3 . At releases of 40000 Millions m^3, navigation deficits are 50%each month, in contrast with a 12% shortage for irrigation. The present minimum navigation requirements are 80 million m^3 per day in the main Nile reach in order to pass ships with large draft depth in the river. Shortages in navigation necessitate preplanning by the MPWWR to allow only smaller draft ships to pass.

Shortages in irrigation demands can be overcome by alterations in cropping patterns, such as replacing high water requirement crops in some areas, such as, rice and sugar cane, by low requirement crops, such as, maize and sugar beets. Rice cultivation currently comprises over 900 thousand feddans, representing 7.6% of the total cultivated area. Water requirements for rice, however, comprise 21% of the total water requirements, based on assumed demands of 8800 m^3/feddan/year. In contrast, maize requires only 2700 m^3/feddan/year. Applying a policy of rice reduction allows application of policies for reduced HAD releases, and an improved hedge on drought for Egypt.

5. CONCLUSIONS

The management runs resulted in finding the optimal release policy from the HAD, in conjunction with increased use of groundwater and reusable drainage water that ensures satisfaction of all irrigation, municipal and industrial, and navigation demands, while allowing increased HAD storage levels. Results indicate that under the optimum conjunctive use policy, overall system efficiency increases from a current estimated 73% to 83%, due to reduction of spills to the sea.

Further reductions in HAD releases from the optimum conjunctive use policy will create shortages both agriculture requirements, as well as navigation. Agricultural shortages can be eliminated by changing the cropping pattern for irrigation demands, such as replacing water intensive rice cultivation with maize cropping with lower irrigation requirements. Navigation shortages may be elinated by allowing ships with smaller draft depth into navigable channels.

Adopting the optimal conjunctive use policy, or the reduced HAD release policies, will increase safe storage in the HAD. This will serve as a hedge on long term drought, as well as increase opportunities for expansion in land reclamation for coping with the mounting population growth in Egypt and the escalating need for food production.

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MULTIPURPOSE IRRIGATION CANALS - GENERATION OF MICRO HYDRO ELECTRIC POWER

Georges-Emile Karam[1] and Ismail Najjar[2]

Abstract

This study describes the experience gained and lessons learnt from a feasibility study on the installation of *ultra* low head (1-3 m) hydro generating sets in irrigation drop structures, covering the existing irrigation schemes in three Indonesian provinces. It attempts to promote the concept of integrated water resources development and use particularly in remote rural communities. Institutional models based on a bottom up approach for a sustainable implementation of projects of that type are presented. The importance of a central coordination mechanism to provide technical support to the beneficiaries, to homogenize the various sectoral policies and to promote and popularize the installation, operation, and maintenance of the micro hydro systems installed in the irrigation channels in underscored. The paper also discusses the interrelationship between the technical and institutional facets and their effect on the success or failure of this technology.

Moreover, the study includes a survey of the international and local market on suitable equipment, their availability and efficiency range. A model for the determination and ranking of potential sites using a Geographic Information System (GIS) was tested in this study.

CANAUX D'IRRIGATION À BUTS MULTIPLES - PRODUCTION DE MICRO HYDRO-ÉLECTRICITÉ

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Résumé

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Cette étude décrit l'expérience acquise et les leçons apprises au cours d'une étude de factibilité portant sur l'installation de groupes co-axiaux (alternateur/turbine) sur de très basses chutes (1-3 m) existantes sur les canaux d'irrigation de trois provinces d'Indonésie. L'étude est en elle-même un effort de promotion du concept de mise en valeur intégrée des ressources en eau et son application, en particulier, dans les communautés rurales éloignées. Des modèles institutionnels basés sur une approche "de la base au sommet" pour un développement durable des aménagements de ce type sont présentés. L'importance d'un mécanisme central de coordination pour assurer le support technique aux bénéficiaires, pour homogénéiser les différentes politiques sectorielles et pour promouvoir et vulgariser l'installation, l'opération et l'entretien des micro-centrales installées sur les canaux d'irrigation, est soulignée. Le document discute également les relations entre les facettes techniques et institutionnelles et leur influence sur le succès ou l'échec de cette technologie.

De plus, l'étude comprend une investigation sur le marché local et international quant à l'équipement approprié, sa disponibilité, sa plage de rendement et son prix. Un modèle pour le choix et la classification des sites potentiels utilisant un SIG a également été testé dans le cadre de cette étude.

1. INTRODUCTION

Micro and mini hydro developments, industries that were booming in the early 20th century, underwent a recession that started in the fifties due to the introduction of the mega hydro industry. Now, with the recent awareness of the ecological hazards of mega hydro developments, these industries experienced a revival in the developed world where programs in France, Portugal and Canada were already adopted by the concerned ministries. In the developing world, this industry is still being introduced in a random manner and it can be safely said that eventhough a few projects have been implemented here and there, the micro hydro industry there is still at a pilot project stage. A benefit that can be drawn from that fact is to use the experience lessons gained from such development to help create a techno-institutional package that will make it viable.

Hydrosult Inc., a consulting company in water resources based in Canada, did a study on *ultra* low head micro hydro potential in drop structures in existing irrigation schemes in the Indonesian provinces of North Sulawesi, Central Sulawesi, and East Nusa Tenggara (NTT). The provinces were visited by the authors and the possibility of introducing this type of technology and its potential uses were researched on the spot.

Some of the study's outputs were:

- Generalized criteria for defining viable micro hydro development at irrigation drop structures;
- partial inventory and classification of viable sites in the project provinces;
- a methodology for the ranking of sites based on type, designs, and standard economic and social criteria with the help of a Geographic Information System;
- technical guidelines for micro hydro installations;
- an inventory of the equipment available locally and in the world market and an identification of their sources, range of operation and efficiency;
- typical layouts of the equipment based on the head range;
- an institutional framework for allowing the design, financing, construction, operation and maintenance, and monitoring of micro hydro plants at irrigation drop structures.

The last three outputs listed above are underscored in this paper. They represent some the most basic prerequisites for introducing this technology to the developing world.

2. EQUIPMENT

2.1- Foreign Equipment:

a survey that covered about 100 companies in more than 30 countries in the world for turbine generating sets that operate at a head less than 3m, which is the head mostly available in the Indonesian irrigation schemes drop structures, was completed. The outcome of this survey produced a list of the most feasible micro hydro turbine-generator sets from a technical and an economical point of view (Table 1). Although the prices might vary considerably from one country to the other and even sometimes between companies in the same country, the technical specifications were, in most of the cases, somewhat similar, especially when it came to the type of runner and generator.

2.2- Local Manufacturers:

Two local turbine manufacturers were identified by the study as follows:

Table 1, Summary of equipment survey

Company name, Country	Address	Fax#	Generator type	Type of turbine supplied	Comments
Ossberger Turbinenfabrik, Germany	postfach 425 Weissenburg i. Bay.	091 41 97720	Induction	Cross Flow or Ossberger	Operates under a head from 1 m to 200 m. and flow from 0.025 m^3/s to 10 m3/s
ITT Flygt, Canada	300 Labrosse Avenue, Pointe Calire Quebec H9R 4V5	1-514-697-0602	Induction	Submersible	They supply 6 sizes of turbines having the following range: head from 2.5 m to 20 m Flow 0.55 m3/second to 12 m3/s
THEE, France	Impasse du Gaz, 54200 TOUL	Fax 83 63 10 98	Induction	Submersible	The manufactured turbines cover heads from 2m to 18m and flows from .2 to 3 m^3/s.
Biwater, England	Clay Lane, Oldbury, Warley, West Midlands B69 4TF	Fax 021 544 3741	Induction	Bulb kaplan and vertical Francis	The manufactured turbines work under heads as low as 1.5 m, but require flows more than 1 m3/s
Canadian Hydro Components, Canada	16 Mian Street, P.O.Box 1105Almonte, Ontario KOA 1AO	Fax 1-613-256-4235	Induction	Kaplan turbines	The manufactured Kaplan turbine operates under a head as low as 2m
Tamar Designs Pty. Ltd., Australia	Foreshore Road Deviot, Tasmania, 7275	Fax 003 94 7565	Induction	Semi Kaplan	This company did not provide details about its machines
Energy Control Systems Limited, Ireland	3 Cranford Hall Stillorgan Road, Donnybrook, Dublin 4,	Fax + 353 1 269 5967	Induction	Propeller Turbine	Turbine available can be used for heads ranging from 1.5 to 5.0 m.
POWERSTREAM, England	1 Riverside House Heron Way Newham, Truro Cornwall TR1 2XN	Fax 0872 222 424	Optional: Induction/ synchronous	Information not available	Turbine(s) available can operate under heads ranging between 1.5 m and 3 m and flows ranging between 0.3 m^3/s and 3 m^3/s
AWT, England	P.O.Box 10 Tetbury Glos. GL8 8RA	Fax 44 0453 834 978	Induction	Fixed flow back -to-back reaction turbine	This company produces turbines covering heads starting at 1m and reaching 10m.and flows from 0.1 m^3/s to 0.7 m^3/s.

2.2.1- <u>HIBS:</u>

A company in Bandung, Hidropiranti Inti Bakti Swadya (HIBS) manufactures cross flow turbines using designs supplied by SKAT of Switzerland. HIBS uses basic tools and local steel to produce the two models, T7& and T12 which operate under a head range of 4-40m and a flow range between 0.03 m^3/s and 2.6 m^3/s. Generators to compliment the turbines can be either induction or synchronous and are supplied by local and foreign manufacturers.

2.2.2- <u>Siemens</u>

The company Siemens manufactures turbines under license from Wasserkraft Volk (WKV) GmbH in West Germany. Siemens has two factories in Java, one in Jakarta and the other is in Cilegon in West Java. Turbines produced by Siemens are of the cross flow type and operate under a head range with a lower limit of 4m.

In the collection of the above data, the project team was faced by many obstacles namely, the difficulty of reaching some countries by fax and that companies, in general, were reluctant to give out technical information.

It can be clearly noticed that all the foreign companies, except one, suggested units that included a reaction turbine of the Kaplan or the Propeller type having a lower limit of operational head of 1 meter while only one company suggested an Ossberger turbine. On the other hand, both local companies only manufacture cross flow or Ossberger turbines that do not operate efficiently under heads lower than 4m. Reverse pump turbines were not suggested by any of the above producers.

Reverse pump and Ossberger turbines can be easily manufactured locally which makes it hard for companies outside Indonesia to compete with the local market. On the other hand, casting of the turbine runners and guide vanes in molds that minimize machining afterwards, which is a technology not used in Indonesia yet, seems to give some companies an edge especially in the speed of manufacturing. Induction generators were suggested by all except one of the companies which suggested a synchronous generator. These available generators will need to be customized by adding capacitors to allow them to operate in isolated locations where they cannot be connected to the grid. The strengthening of the rotor is also required because these generators are not usually designed for runaway speed conditions. The quotations received also included electronic load controllers in the generating units which have proven their effectiveness in developments of this size. The average power coefficient of all the companies products was calculated as 7.50, and the value of this coefficient and its variation between the individual companies is presented in Figure 1. Locally manufactured turbines were noticed to have a power coefficient of 15% to 20% less than foreign turbines.

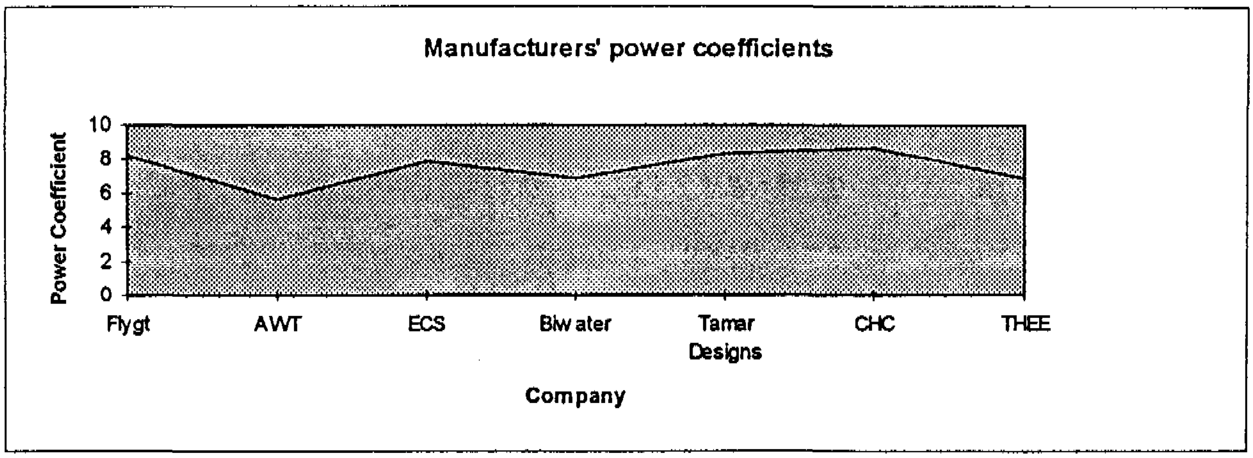

Figure 1, Foreign manufacturers' power coefficient

2.3- Equipment installation and general layout

In micro hydro installations, the main equipment to be installed are the turbine, generator, electronic load controllers and draught tubes while the penstock is, of course, usually very short due to the low head. Although the speed of the water entering the turbine is low and this minimizes the risk of silt abrasion, a settling basin is recommended to be installed upstream of the intake structure. The study also recommends generalized layouts of the equipment that would best suit the nature of the irrigation schemes design in the provinces of study. The layouts that are suggested by this study are meant to least modify the existing structures and their original function. Periodic maintenance of the irrigation structures which requires the flow to stop for some time, was a problem for the micro hydro application. Such a problem can be overcome by installing hybrid systems using diesel as the alternative. The typical layouts that were recommended by the study are presented for two head ranges in figures 2,3 and 4.

3. INSTITUTIONAL FRAMEWORK

The study recommended an institutional framework for allowing the design, financing construction, operation and maintenance of the micro hydro developments. The survey conducted by the team indicated that most of the people interviewed in remote areas, where the electric grid was absent, were willing to try micro hydro, if its cost for lighting was cheaper than kerosene, and if it would allow them to indulge in some of the luxuries of life such as using electrical appliances.

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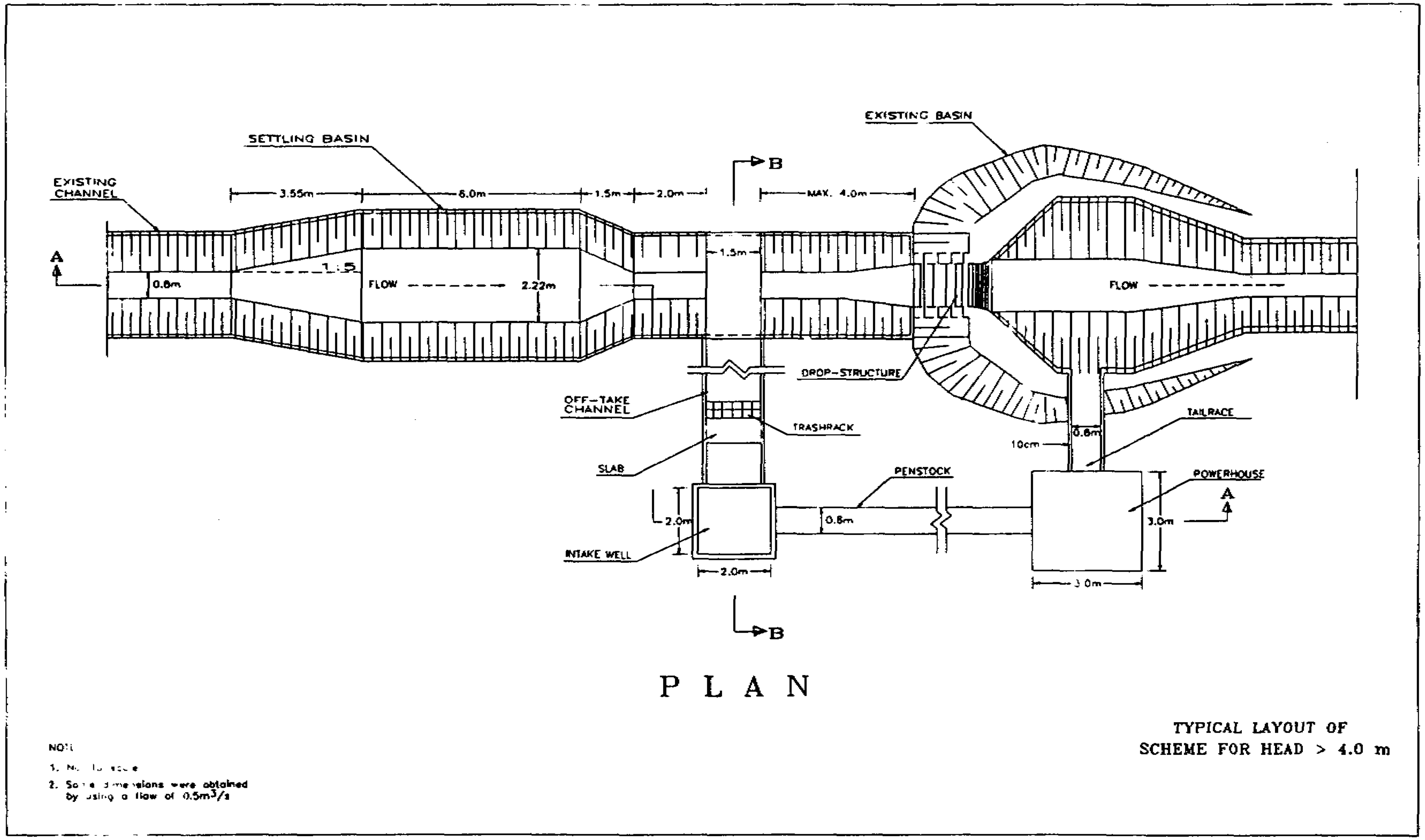

Figure 2, Plan view of scheme for head > 4 m

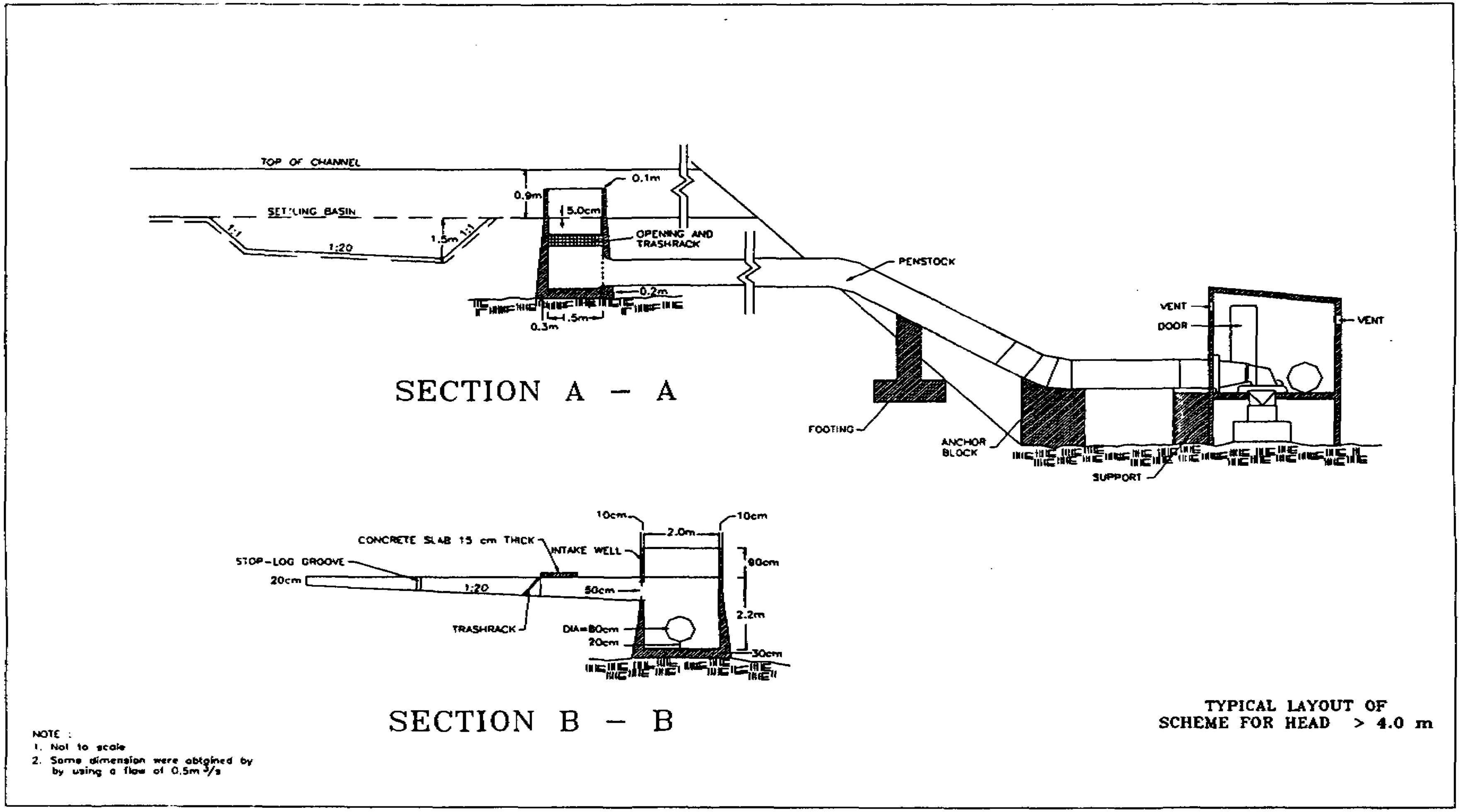

Figure 3, Cross-section of scheme for head > 4 m

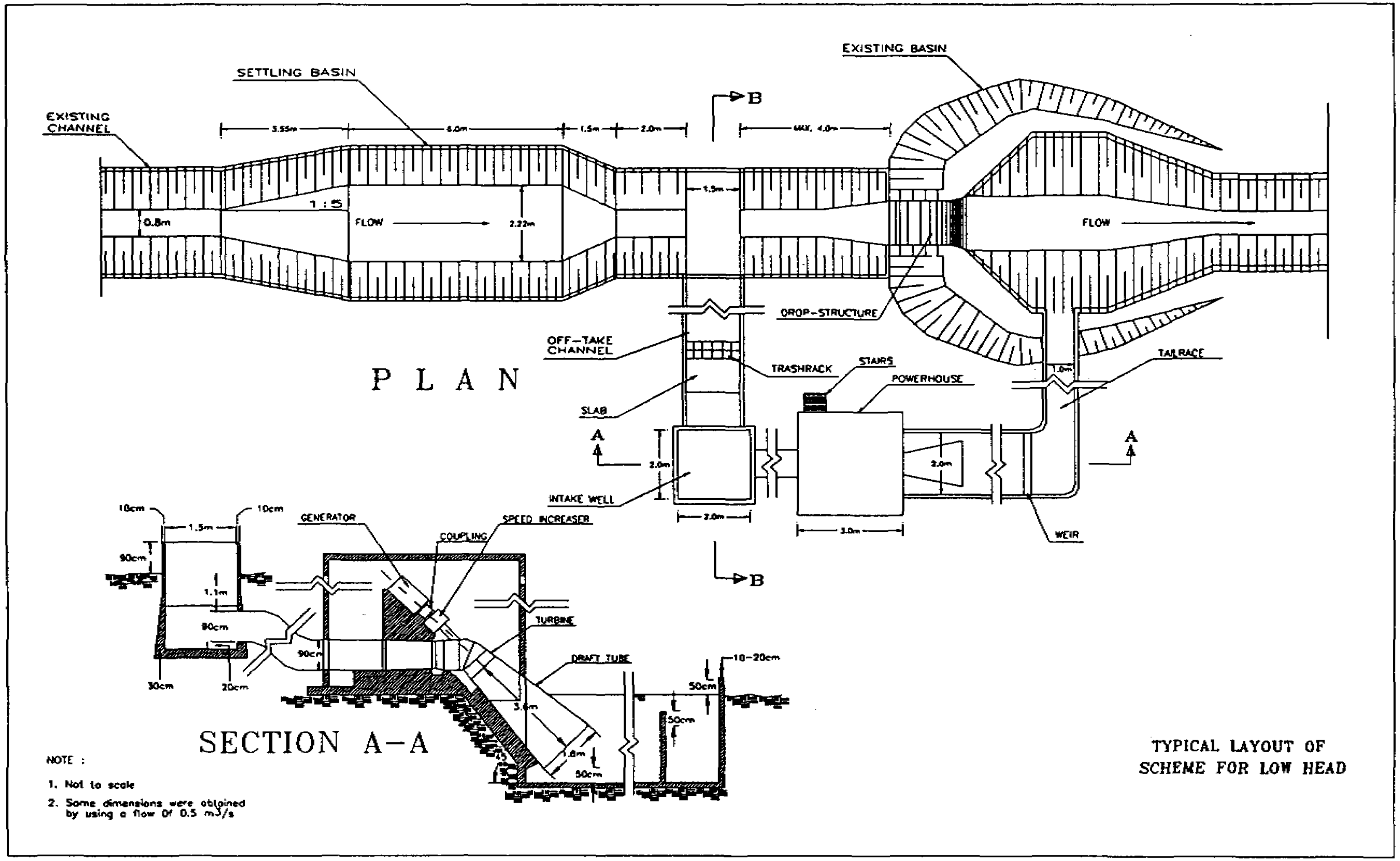

Figure 4, Plan and cross-section of a low head scheme

In attempting to recommend a community based institutional arrangement for micro hydro installation, the team studied cases in neighboring countries that have longer experience with this source of energy.

Lessons learnt from the Nepal, Papua New Guinea, experiences mainly underscore the importance of a bottom up approach for planning and implementation of the systems and the necessity of encouraging local industries in the field of micro-hydro. Locally fabricated generators, although sometimes not as efficient as some of the European or North American ones, are still more advantageous. However, it was reported that in many instances in Nepal the desire to accelerate the country's development process has repeatedly prompted the Government to introduce imported generators provided by donors under the tied aid principle. This has caused the closure of local industries and the loss of local expertise, and know how, which has been developed since the early sixties.

The lessons learnt from the Papua New Guinea experience also highlight the importance of a community based approach and the use of local materials and skills. Moreover, the Papua New Guinea experience indicates that a community based construction procedure should also be adopted rather than a conventional procedure. The community-based approach should use day labour for the skilled laborers to be supervised by an experienced and motivated volunteer acting as a project manager. The bulk of the unskilled work should be done by volunteers. Such an approach, it was estimated, can save the project up to 25% of the construction cost.

Other constraints to the development of micro hydro in developing countries as depicted by the experiences in Nepal, Papua New Guinea and Indonesia, are the lack of financial and technical support of the local technology.

4. ROLE OF GOVERNMENT IN PROMOTING AND SUPPORTING MICRO HYDRO DEVELOPMENT

As much as it is advisable to decentralize micro hydro developments to the level of users, the Government can still play an important role in sensitizing the population, promoting the advantages of the technology and providing technical assistance.

The present study, revealed the fact that one important input from the Government would be the preparation of an exhaustive inventory of candidate sites, to be made accessible to the potential users/investors. The inventory would include data on the head, flow, operation protocol of the irrigation canal, and the generic adaptable designs and specifications of turbines.

Moreover, the inventory would also include a demarcation of areas covered by Grid of the state owned electric company. This will optimize the number of sites to be investigated. The monitoring of actual water levels and flows over a cropping period to assess the need for introducing a hybrid diesel system would also be required. The creation of a special division within the Ministry of Public Works (micro hydro division) is thus recommended.

Moreover, the Government should promote the development of locally fabricated turbines through encouraging joint ventures between local and foreign industries, and in cases of foreign aid programs encourage the donors to provide locally fabricated sets.

The institutional arrangements recommended by the study included provisions for cost recovery through a tariff system, credit facilities and education and training.

The procedure for developing micro hydro for rural communities would start with a promotion campaign by the responsible ministries at the central and provincial levels, followed by consultations among Ministry of Cooperations, Ministry of Public Works and the state electric company (PLN), and candidate villages targeted for micro hydro development.

5. AN INSTITUTIONAL MODEL FOR DEVELOPING COUNTRIES

The existing situation regarding the rural electrification programmes in Indonesia involves at least six agencies with the Ministry of Mines and Energy and the PLN (state electric company) taking the lead. However, PLN's policy is not to get involved in any hydropower development of less than 200 kW, thus micro hydro is out of PLN's interest domain. Moreover, the control of physical structures and the use of water in the canals for running the tribunes rests with the Ministry of Public Works. Thus, the study suggested that the mandate for developing canal micro hydro should be given to the Ministry of Public Works (figure 5).

As emphasized above, development of *ultra* low head micro hydro in irrigation canals or small streams can only be sustainable if they are planned, operated and maintained within a community based institutional arrangement. The study revealed the fact that willingness of the communities to accept micro hydro development is dependent on the affordability by the users of the technology and also dependent on the type of downstream uses.

The experiences in other countries, and the socio-cultural survey conducted in three provinces in Indonesia, allowed the study team to formulate an appropriate institutional model which takes into consideration most of the constraints to successful micro hydro development. A concept for an investment/credit facilities for potential investors was also elaborated.

The model accounts for the following aspects:

1. Respect for the existing legal and institutional frameworks of the agencies involved;
2. Community based planning and implementation, "bottom-up decision making process" with government technical support;

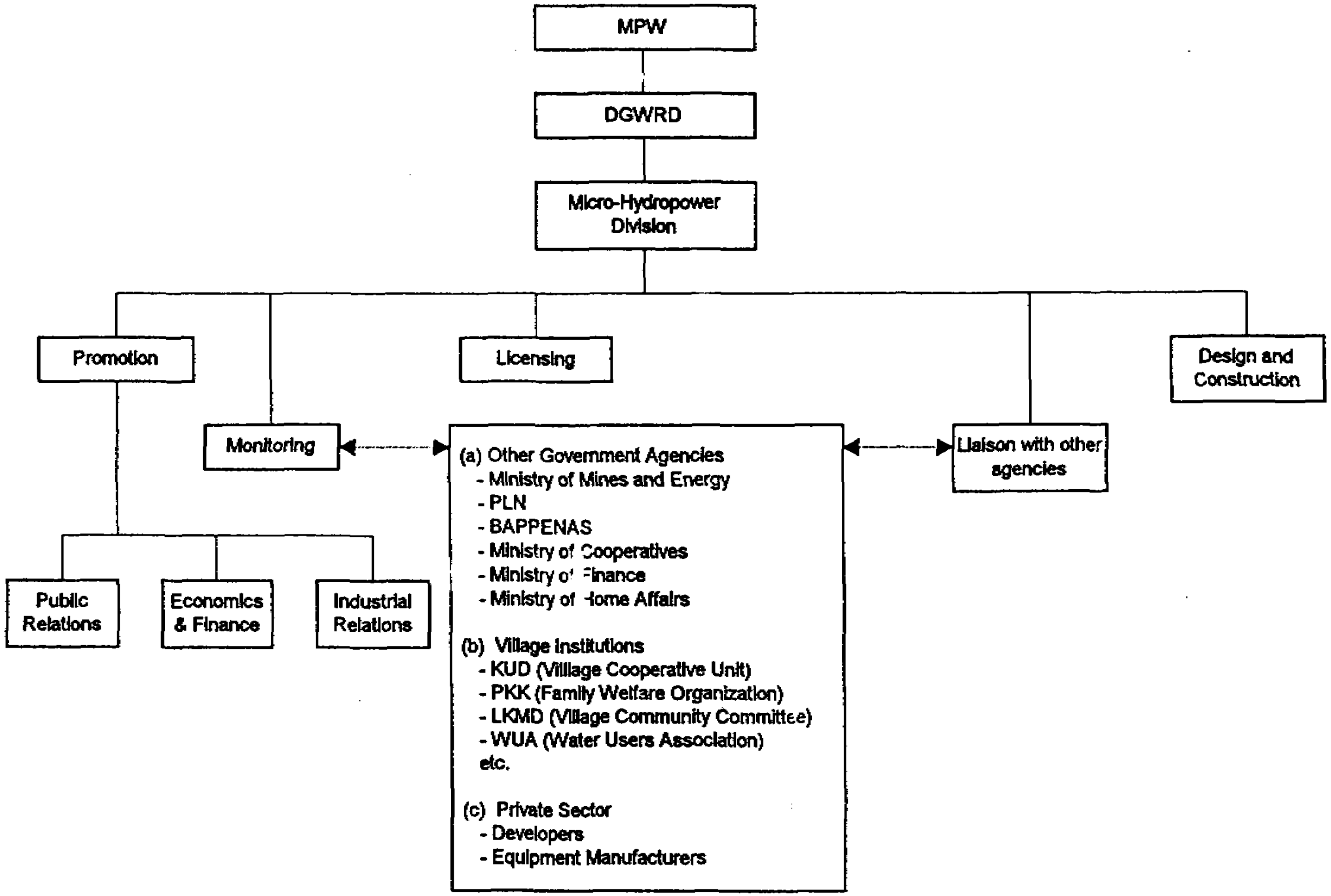

Figure 5 - Organizational Chart

3. Community acceptance and willingness;

4. Financial affordability of the given scheme, and access to financing from a special bank set up for micro hydro development, or through a private entrepreneur;

5. Protection and presentation of land ownership of the micro hydro project site.

6. Uncomplicated and preferred procedures for licensing, construction, operation and maintenance by the community.

The process of selecting a rural community as a target for micro hydro development, starts with the inventory, and the socio-economic survey which determines the candidate villages through a process of technical, social and economic ranking and screening (figure 6). Once a village is targeted, it is important to initiate discussions at the village level to determine the willingness of the community to embark on this activity and also to explore the community's degree of affordability of the system, as well as the expected use and benefits. Once these factors are clarified then the issue of tariff fixing and financing is raised. Financing of village micro hydro would then be obtained through a credit facility from a bank or a community fund raised for that purpose.

A banking facility that would be partly owned by the community and partly by the local government was recommended. The bank would make small loans to low-income groups on the basis of trust and confidence, without any collateral. The loan is made after potential borrowers are organized into groups, given training in managing the loans, and later assessed as being of low risk.

6. SUMMARY & CONCLUSIONS

A study was done by Hydrosult Inc., of Canada to determine the feasibility of installing *ultra* low head micro hydro generating sets in existing irritation channels in Indonesia.

The study covered, the technical, institutional, social, and economic aspects of the problem. An international survey of the equipment available showed that only a few companies specialized in producing equipment that operated at heads lower than 4m. Local equipment was limited to cross flow turbines of medium to poor workmanship. For this reason, the micro hydro installations that were visited by the project team and that were using locally manufactured equipment were all operating at heads above 4m. The workmanship of the local equipment reflects on the overall efficiency, however, even at an efficiency 15% to 20% lower than the average efficiency of similar foreign equipment, local equipment was found to be more desirable to private developers due to its low cost. To make *ultra* low head installations more viable, a local industry of high quality reaction turbines must be encouraged through a joint effort by the public and private sectors.

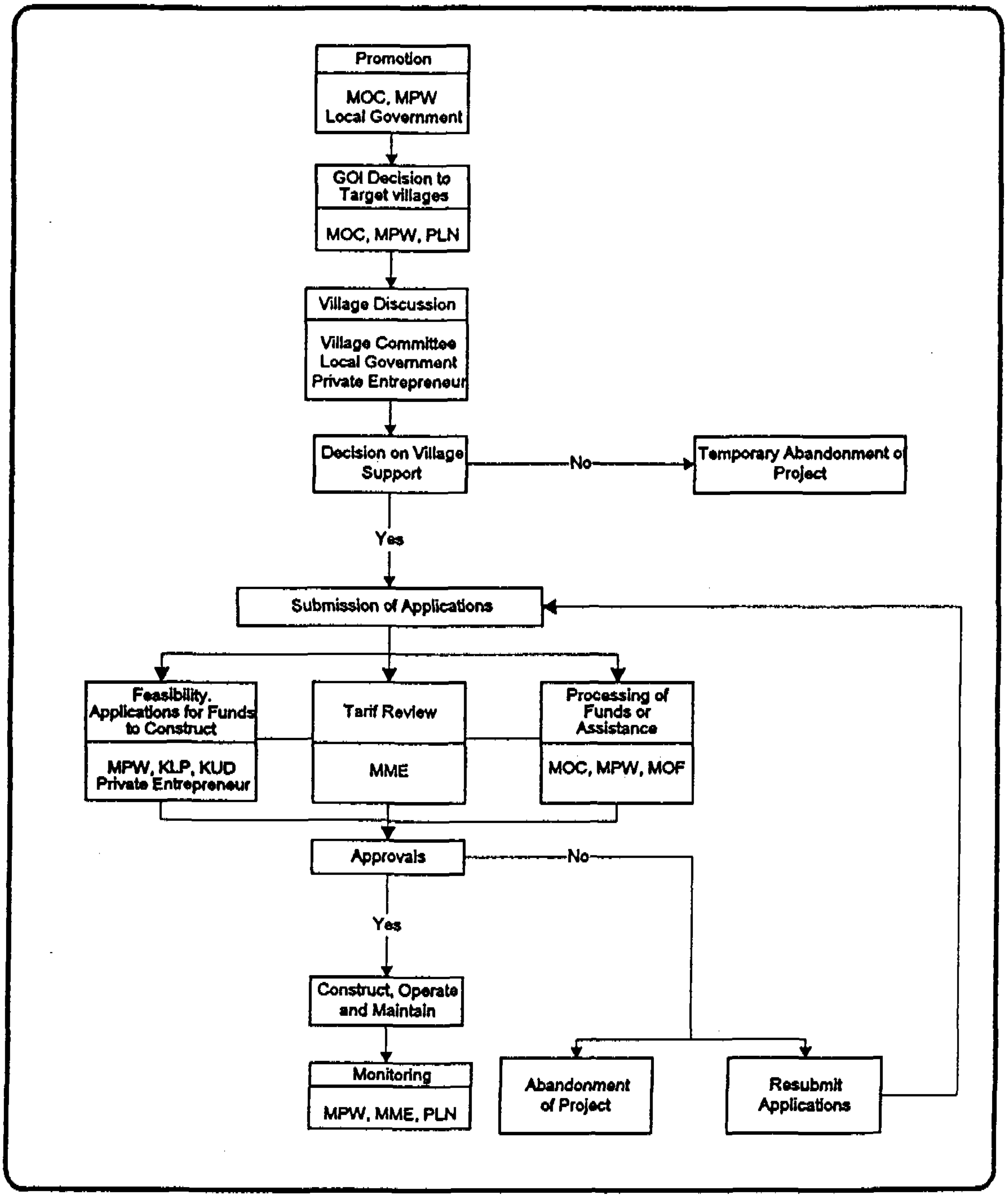

Figure 6 - Flow Chart of Administrative Process

The study concluded that the financial state of typical rural communities indicated that they would be able to pay for the connections of micro hydro systems to their houses but that they could not raise the capital to construct the micro hydro station without external help. However, the terms for lending development capital for construction of micro hydro stations must be reasonable and compatible with the affordability level of the potential users. A concept for a banking facility relying on trust and confidence rather than on collateral for providing loans to low income borrowers has been suggested.

In as far as the institutional framework for micro hydro is concerned, the team has recommended a model that respected the existing legal and institutional arrangements of the existing agencies. The role of these agencies within this framework would be one of promotion, technical guidance and financial facilitation. The development strategy of micro hydro should be based on a bottom-up approach and on a quick, uncomplicated support to the developers, whether entrepreneur or a group of individuals from the community.

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METHODS OF COMPUTATIONS AND FORECASTS OF WATER RESOURCES FOR DEMANDS MANAGEMENT

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ABSTRACT

A management of water resources is based on a knowledge about state and dynamics of river runoff in present time as well as in a future. The main aspecial feature of modern period is direct (human activity in the river basins and river channels) and in-direct (modern climate change) anthropogenic impacts on thenatural water resources. The detail algorithm and scheme of hydrological computations and super-long-range forecasts for in-homogeneous runoff time series is considered as well as for different informational conditions of hydrological, climatic change and human activity data. New methods are offered for estimation of complexity of the process; for separating of complex process into simple and homogeneous constituents; for formation of the models of time-series; for expert estimation and forecasting of nonstationarity constituents and for hydrological computations. Application of new methods of assessment of time structure of long-term runoff variations and other factors and methods of hydrological computations and forecasts for demands management is shown.

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1. STATE-OF-ART OF THE PROBLEM

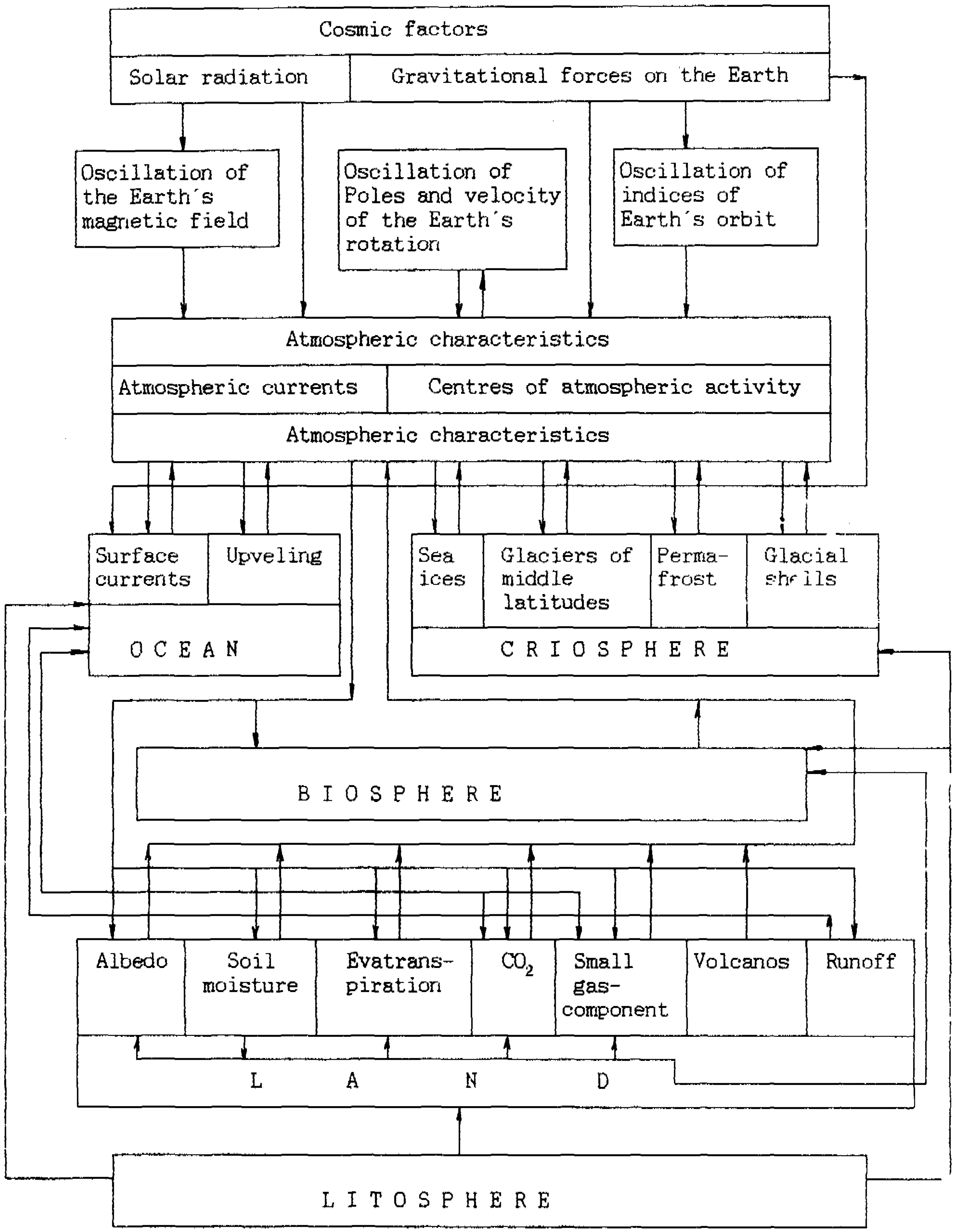
A rational use of water resources, including of their management, is based on a knowledge about a state and a dynamics of river runoff in present times as well as in a future. Nowadays there is a contradiction between existent methods of runoff description in the form of a distribution function or generalized parameters, which are transfered from present to future, and changing external conditions (anthropogenic climate change and impact of human activity factors) as well as an demands of a practice for using of the dynamic properties of water resources such as: series and periods of water raising and water decreasing, an extraordinary events, etc.

Another contradiction takes place in modern hydrological computations between conceptions of computations and super-long range forecasts. Methods for hydrological computations are based on the concept of a random autocorrelation sample, while the methods of super-long-range forecasts are based on the deterministic and deterministic-stochastic laws of long-term runoff series over the time. At the same time. the hydrological computation is a probabilistic forecast of extreme events appearance in given time interval in the future (return period). Furthermore, hydrological computations and forecasts are based on the same information, therefore they have to use the same model of the time series.

Many publications dedicated to the proof of the modern climate change, for example [1,2,3]. It has been accepted that climate is subject to changes, therefore, those climate characteristics which have been observed during the recent 100-200 years should be studied and explained. The review made on the state-of-art of this problem and on the interrelations in the hydroclimatic system shows that long-range variations in the climate may be explained both by natural and anthropogenic reasons.Since climate models are far from being perfect, the results of estimating probable changes in runoff may be variable and may differ by an order. In majority of studies it is noted that the most realistic changes in runoff in future should not exceed 20-30%. The empirical data analysis shows a rise of air temperature in the Northern Hemisphere and ambiguous conclusions on the assessment of trends in some particular series of climate and runoff. It is obvious, the climate change have to impact on water resources, the problem - when it'll begin to impact or began already and how much is its impact in the runoff fluctuations. The complication of natural picture with many intercommunications in climatic (hydrometeorological) system is shown on Fig.1.

There are many publications dedicated to the problem of investigation of historical time series of annual runoff, which characterizes of water resources. But the results of investigations are essentially different and depend on applicable methods. Conclusions about stationarity of historical fluctuations are obtained, as a rule, with the help of the statistical methods, which have hypothesis of the stationarity or homogeneity in their base. So, this hypothesis is necessary to refute with very high level of reliability or probability (P> 95-99%), that makes it very firm, although a difference between two or more values mounts to some tens percentages and it's important for practice. At the same time, a non-stationarity of historical fluctuations of water resources are proved in many investigations even in those, which are based on statistical methods, for example [4,10,13].

Figure 1. Main intercommunications in hydro-climatic system



Then, short review about state-of-art of the problem is evidence of a necessity of the transition from analysis of a generalized statistical parameters of water resources time series to the investigation of their dynamic properties. This problem is existing now and it is necessary to consider and to decide if not today, but tomorrow with more efforts.

2. ASSESSMENT OF NON-HOMOGENEITY OF HYDROLOGICAL PROCESSES

A structure of hydrological historical time series defines all external influences on river runoff. In this paper three another proofs are proposed for assessment of complexity of hydrological processes.

A. Empirical verification of the justifiability of generalized parameters (mean values and standard deviations) of hydrometeorological time series is carried out corresponding with Mizes approach in statistics and theoretical formulas are derived in application to the Markov autoregressive model, as the most commonly used in hydrometeorology. Approximation with these formulas of the series of the mean annual runoff, precipitation and air temperature for all former USSR revealed that present-day models are inadequate to properties of empirical series. Relative differences between the empirical and the theoretical models grow with an increasing size of observational data and this result is contrary to the main concept of mathematical statistics: the greater time series size, the more reliable general parameters are obtained. The cause of such type of distinction, as it's proved in theory, is connected with compositional nature of processes, which consist of two components at least: random component and any type of trend-component [6].

B. Empirical verification of the significance of long-period trends in annual runoff time series has been executed for sites with long observation period in some regions of former USSR. Linear trends in historical time series become significant from sample volume of 20-30 years for river basins which are in conditions of the intensive human activity and from sample of 60-70 years for river basins with low level of human activity. The conclusion of this investigation gives the fact that factors of man's activity are more significant for short time period. In others cases impact of climate changes on natural river runoff became statistical significant from long period only, but they take place really [4].

C. Empirical verification of the complexity or composite nature of runoff time series has been fulfiled using new empirical method. Main idea of the method: a comparison of sample range for random autoregressive model (Rm) with the range of real hydrological series (Re). The process is homogeneous if Re < Rm and the time series contains more then one process, if Re > Rm. New index is used for the non-homogeneity part or contribution of the complexity in whole process (in %):

$$S1 = (1 - (Re - Rm)/ Rm) * 100\% \qquad (1)$$

Results of investigations are shown that maximum index S1 for many regions of European part of Russia is 50-70% and mean value of index is fluctuating from 15 to 35% for different regions.

3. THEORY

The concept of cyclic nature of fluctuations of natural processes (an annual runoff too), is the first basic property of hydrological processes. The cyclicity is a sequence time series of phases of raising and decreasing of water with different periods:

$$Yi < Yi+1 < ...< Ym > Ym+1 > ...>Y m+k < Ym+k+1 < ..., \qquad (2)$$

where m - period of water raising, k -period of water decreasing,
m+k - period of the cycle.

The cyclic nature and repeatability of earthly processes are related with the rotational movement inherent to all objects in the Universe (the Galaxy, the Solar system, the Earth, the atmosphere, etc.). Periodicity (the constancy of cycle periods in time) of the dynamics of space objects is refracted to the cyclicity and rhythmic (non-constancy of cycle periods in time) for hydrometeorological processes on the Earth. Another reason of cyclicity is a property of inertia and entropy of the system, when after an influence of external factor, the system speed in former state with minimum of internal energy. The second basic property of complex natural processes, and runoff too, is a composite nature of their structure, that is conditioned by influence of different scales factors, forming the runoff process as a whole. As a rule, it is not known a priori how many simple homogeneous processes participate in the formation of investigated hydrological time series, or at least how many different-scale components, each being also a composition of similar-scale processes, represent the time series.

Considered historical time series of annual runoff in modern conditions may consist of three components at least, such as: natural component, component is connected with direct human activity and component, is connected with climate change. Such components may be more, if some of them are a non-homogeneous, especially it's concern of natural component.The following dynamic characteristics of the process are taken: a coefficients of the cycle function when a full description of the process take place or the main information characteristics of dynamics such as periods and amplitudes of the cycles.

Proposed methods for analyse of a cyclicity and a composite structure have in its base two theories: the theory of random function and the theory of an experiment.The first theory defines a selection of realizations of random function such as: cycle of historical fluctuations or hydrological event. The second theory defines an algorithm of the mathematical model of the nonhomogeneous composite process, using an experimental data. The main thesis of such approach:it is not known a priori that class of the models our investigated process belongs to: stochastic, deterministic or stochastic-deterministic and this problem may be solve from the analysis of correct chosen dynamic characterictics only.

4. ACCEPTED CONCEPT FOR CLIMATE PRESENTATION

In general, climate is a statistic assembly of weather conditios [2]. In turn, the statictic assembly is characterized by some generalized indices of selected synoptic parameters.

The main problem is that the collection of synoptic data is extremely laborious, since it is related to synoptic charts processing. A new approach is proposed in this paper to determine synoptic parameters directly from the observation series on air temperature which simplifies the procedure of this determination and extends the series. The main idea of the proposed approach is in the following: if one is at a point (at the station) it is possible for him to determine the air masses transport (synoptic formations)on the basis of a sudden leap in air temperature. Moreover, these masses are displayed in air temperature series as pulsations. Each air mass is unambiguously characterized by the following factors: air mass duration at the point, mean and extreme temperatures inside the air mass, temperature variations, etc. When making a statistic or functional generalization of the synoptic parameters during a year and when introducing new factors (amount of air masses, frequency for a particular interval, etc.), we transfer to a new level of characteristics - i.e. to the climatic level. When analyzing the climate charac-teristics for a long-term period, it is possible to estimate the climate stability. If compared with the routine approach, where series of mean annual air temperatures, total annual precipitation, etc. are analyzed, the present case reveals the circulation capacity of the atmosphere which is responsible for climate changes.

5. ALGORITHM OF THE APPLICATION

Rational use of water resources are defined by an information about computational or forecasting hydrological characteristics. It is obvious, if will be low water during some years in future, it is necessary to envision of water accumulation in reservoires, when high water takes place. Another case, water users can take a water from neighbouring river, if they are fully confident it'll stay enough in future as well as at present time. Many others examples could be given to prove that a knowledge about runoff properties is a base of water resources using.

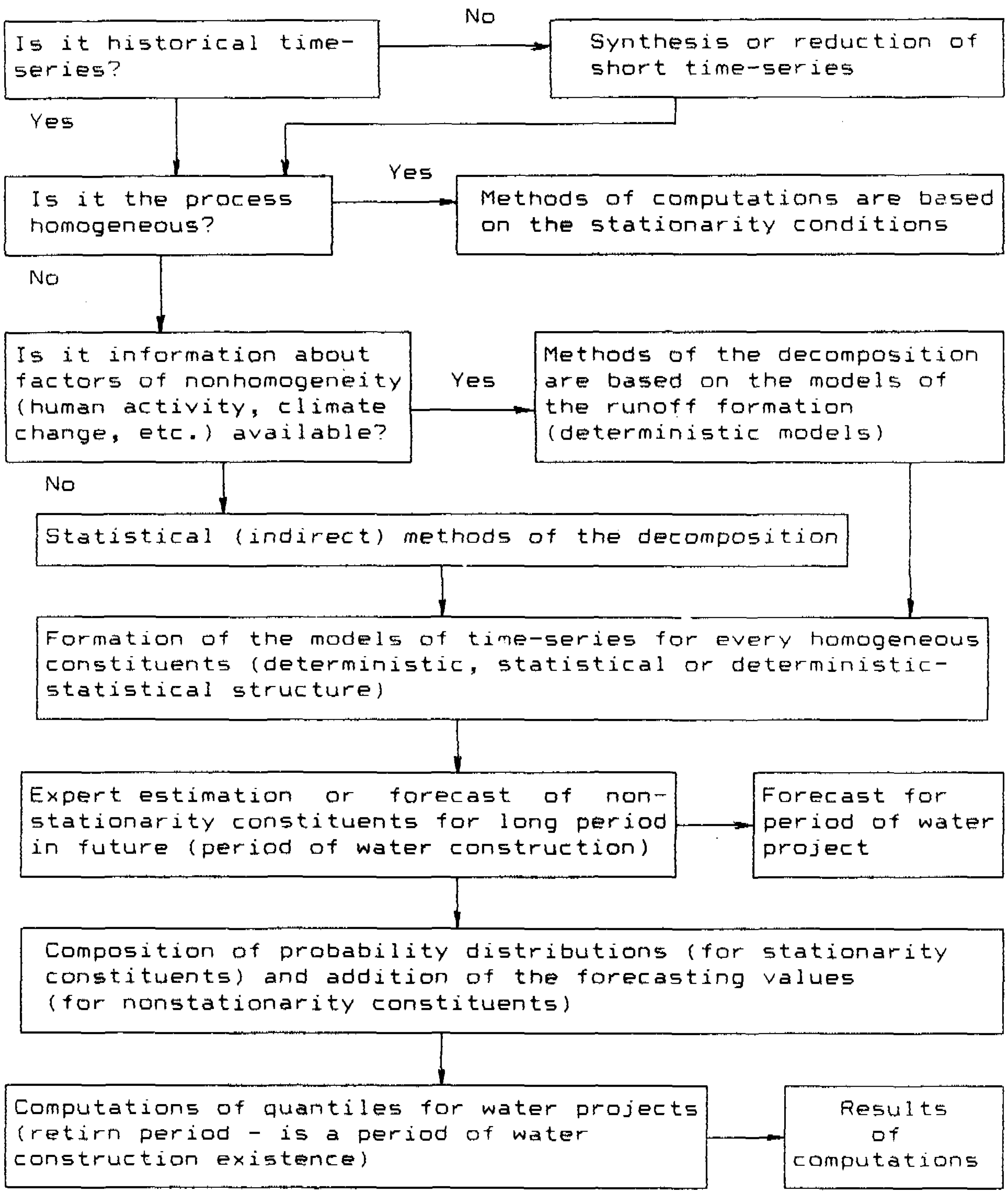
Offering scheme of hydrological computations and forecasts for conditions of the climate change and human activity are given on Fig.2. A setting of its main units are as follows.

5.1 Estimation of sufficiency of time series intends for its reduction and filling of data absence if it's possible, so the longer time series, the more reliable of runoff computations. In some river sites, where the data are absent but for the practical goals it is necessary to estimate of water resources, a synthesis of historical time series is made. This unit are realized only for natural conditions or for the same level of human activity and climate change over the area.

5.2 Analysis of the homogeneity of the time series consists of the estimation of the informative reliability, an analysis of the extraordinary events,the estimation of a composite of the process and some other properties. This unit is necessary for elimination and correction of unreliable data and for estimation how many quasi-homogeneous components form of analysed time series. The existing statistical methods of the computations could be used, if is established that the process is homogeneous and has no time dependences.

5.3 An availability of the information is estimated about factors which upset the time series homogeneous and lead to the composite of the process. This unit is necessary for

Figure 2. Algorithm of the methodology of computations and forecasting of runoff with changing natural conditions



choice of the decompositional way. If information is available over the historical period, runoff formation model could be used, where every quasi-homogeneous components are considered as a part of common runoff or as a complex of the factors.

5.4 Statistical or indirect methods of the decomposition are used if information about factors is not enough, unnumerical or is absent at all. These methods are used as for hydrological time series, as for historical time series of the factors of annual runoff (air temperature, precipitation, etc.) to separate the impact of climate change and human activity.

5.5 Mathematical models of time series are determined as for every component as for the whole nonhomogeneous process. The class of the model of the process (deterministic, stochastical or deterministic-stochastical) defines the subsequent ways of its application for goals of the forecast: probabilistic, deterministic or mixed.

5.6 The estimation of the possibility for forecasting of the separate components are realized on the period as water projects, as waterworks exploitation period.

5.7 Hydrological computations are executed and quantiles (possible extreme values) are obtained from the composition of stochastic components and an addition of the forecast of deterministic and mixed components.

6. USING METHODS

The particular cycle of the methods are used on every stage of the application. So there are very many methods, a short setting, main idea and publicational reference could be given only. The following methods are applied in conformity with stages of the algorithm.

6.1 For filling of data absence and for reduction of time series are applied methods, based on the methodology of space relation-ship for hydrological historical data as the meteorological processes (main factors of runoff), generalised for an year, are homogeneous over large area. Methods of data reconstruction are based on the regression relationships with more longer historical time series at analogue-sites, if period of the observations of short time series $n > 5\text{-}7$ years (long-term synchronous). Group of methods based on the relationships with analogues for particular years are applied if period of short time series $n < 5$ years (yearly synchronous). If data are absent at all, their synthesis are realized by many methods: the construction of the models of runoff formation, methods of the regional analysis, interpolational maps of annual runoff for every year, etc.

Offered method of runoff synthesis for case of annual runoff is based on the relationships between runoff and its meteorological factors and on the runoff formation model of balance structure, the coefficients of which are determined by least square method (LSM). The peculiarity of the model this fact that annual runoff is considered as non-homogeneous process during the year according to conditions of its formation (genesis) and consists of some homogeneous seasonal components. For many regions of Russia homogeneous seasons could be chosen in a year, they are: spring snowmelt flood, summer-autumn rainfall floods and winter low-flow. In every case, runoff is formed by it's own particular complex of factors.

Algorithm of the method of annual runoff synthesis consists of following stages.

- a choice of long runoff time series and its factors at analogue-sites in the region where site of future water project is located;

- homogeneous seasons are determined and volumes of runoff and its factors are calculated for every seasons;

- equations between runoff and their factors are determined for every season and the best of them with common structure for whole region are chosen;

- dates of the beginnings of seasons are generalized over the region and are transfered to the investigated catchment;

- coefficients of chosen equations for every season are generalized over the region and are transfered to the ungauged site;

- time series of seasonal runoff at ungauged site are synthesized using meteorological data over this basin and transfered coefficients of equations;

- time series of annual runoff at ungauged site is defined as the summation of seasonal runoff.

New methods were realized for separation of streamflow hydrograph into homogeneous seasons and for determination of dates of season beginnings, they are: trancation procedure, using hydrological data only and complex method, using meteorological data of daily temperature and precipitation. The most important step of runoff synthesis is a determination of the effective structure of the seasonal runoff equations for the region. This step has following algorithm:

- assuming factors for every seasonal runoff are chosen;
- a type of one-factor relationships is determined;
- a priori assuming structure of seasonal equations is chosen: a linear additive structure, a non-linear additive structure, a linear multiplication structure and a non-linear multiplication structure;
- method is chosen for construction of regression equations and for computation of their coefficients: a stepwise procedure, an elimination method and using of given list of factors;
- the most effective equation is chosen from 12 constructed equations (four different structures of equations and three methods of coefficient construction) for every site in region by optimization from some tests: maximum correlation coefficient of equation, minimum number of regression coefficients and minimum standard error of equation;
- the most effective common structure of equation of every season is chosen for region, using a principle of maximum recurrence of effective structure over space.

Generalization of dates of beginnings of hydrological seasons is realized by methods of equal date lines and meaning for region. The same methods are used for generalization of coefficients of regresion equations and besides that the relationships between coefficients of equations and basin factors are considered.

Application of the offered method is shown for case study in the cold region of North of European part of Russia. Five basins were chosen as ungauged sites for runoff synthesis. More then 40 river basins with historical time series were chosen for construction of equations between runof and their factors for every season. As a result, the following equations of common regional structure were obtained:

for spring snowmelt

$$Y = b1^*S + b2^*Xss + b3^*Uaut + b0, \text{ with mean } R=0.82, \qquad (3)$$

where S - maximum snow supply, Xss -precipitation of snowmelt season, Uaut - soil moisture before winter;

for summer-autumn season

$$Y = a1^*X^3 sa + a2^*X^2 sa^*D + a0, \quad \text{with mean } R=0.77, \qquad (4)$$

where X sa - precipitation of summer-autumn season, D - average deficiency of moisture for summer-autumn season;

for winter low flow season

$$Y = k1^*X^2 sa + k2^*n + k3^*Xsa^*Tw + k0, \text{ with mean } R=0.60, \qquad (5)$$

where n - duration of low flow season (in days), Tw - average air temperature for winter low flow season.

Using these equations and generalization of dates of the beginnings of seasons and coefficients over the region, they are transfered to the ungauged sites and synthesized seasonal and annual runoff. Comparison observed values of seasonal and annual runoff with computed values is shown that standard error for spring snowmelt is 24%-48%, for summer-autumn period - 20-24%, for winter low flow period - 13-33% and for annual runoff - 8-14%.

6.2 Complex of the statistical methods and methods of runoff genesis are applied depending on the informational availability. The methods of runoff genesis have more priority than statistical ones. Statistical methods include in themselves different mathematical criteria, corrected for the peculiarities of hydrological information (natural intra-seria relationship and a skewness of the probability distributions) [11], and methods of complex process estimation are based on the signal/noise ratio, value of the trend-component in time series and on verification of the justifiability of generalized parameters [6]. In fact, all methods give a qualitative conclusion only. It could be said about homogeneity or composite of our investigatied process, but it is not known how many different-scale quasi-homogeneous components represent the time series.

6.3 Decomposition of the complex process, if information is available about factors of non-homogeneity (climate change, man's activity, etc.) over the historical period, is based on the building of the runoff formation models of balance structure and them coefficients are determined by LSM (least square method). The peculiarity of the model this fact, the annual runoff are considered as nonhomogeneous value, so the conditions of its formation are not homogeneous during the year. Therefore, annual runoff is a sum of seasonal runoff for homogeneous periods, for example for Russia area, such as: spring snowmelt flood, summer-autumn rain-fall floods and winter low-flow [7].

6.4 There are many situations, as a rule, when information about factors of non-homogeneity is absent for historical period, but it is known a composition process takes place and have to do a decompositon. The existing decomposition methods (spectral and correlation analysis, different methods of smothing, etc.) have many restrictions to the kind of the cycles function and their parameters. They require a greater a priori information on the model structure of the time series which is usually unknown. Hence, new decomposition methods have been developed, such as: a truncation method and method of the poli-linear decomposition, which assume a priori only the assembly of the composition structure according to superposition principle [8,9]. It means, that any interaction of two adjacent scales is reflected in the coefficients of the additive structure:

$$Y = b1Y1 + b2Y2 + B0 \qquad (6)$$

$$B1 = f(Y2), \qquad (7)$$

where Ty1 > Ty2, T - the process period, Y1 - large-scale process, Y2 - smaller-scale process.

The algorithm of the truncation method is in the following:

- all the extrema of the process are determined first;
- then only those extrema are selected (the other ones are omitted) which are related to the process of the least scale according to the condition that the amplitude of their cycles exceeds the amplitude of the error:

$$Ymax = Y'max, \quad Ymin = Y'min, \quad \text{if } Aa > Ae, \qquad (8)$$

where Y'max, Y'min - all process extremes; Ymax, Ymin - extrema of the smallest scale process A; Aa, Ae - amplitudes of the smallest scale process and errors;- the minima of the least scale process are connected by lines the shape of which corresponds to the function of the cycle of the next greater-scale process; and differences are determined between the observed data and those lines characterizing "a pure" process of the least scale with errors:

$$Z1i = Yi - Y'i, \qquad (9)$$

where Z1i - "truncated" process of the smallest scale, Yi - observationed data, Y'i - the rest part of the process Y; - then the procedure is repeated to select the process of the next scale, and this is repeated until 1 or 2 cycles of the grea-test-scale process are left; The main idea of the method of poli-linear decomposition of the fields and processes - a factoral decomposition. An example of common structure of such model for hydrometeorological processes:

$$Y(X1,X2,X3,X4)=C0(X4)+C1(X4)*Yp(X1,X2,X3)+E(X1,X2,X3,X4), \qquad (10)$$

where X1,X2 - geographical co-ordinates,
X3 - number of the month,
X4 - number of the year,
C0,C1 - coefficients are obtained by LSM

For practical cases:

$$Y-Yp = (Yc-Ypc) + By*(Yp-Ypc) + Ey, \qquad (11)$$

where Y - field of the hydrometeorological characteristic over the area in j year,
Yp - the middle field of the hydrometeorological characteristic for historical period,
Yc - the mean value of the hydrometeorological characteristic over the area in j year,
Ypc - the mean value of the hydrometeorological characteristic over the area and for historical period,
By=C1-1, A=Yc-Ypc

6.5 There are two ways to make a model of a process or an event. In the first case the model structure is specified a priori from theoretical assumptions and coefficients are determined from experimental data. Despite of the merits available, this approach may lead to great errors if the specified model structure does not correspond to a reality. The other approach, which is used in this paper, is based on the methodology of the theory of an experiment and it's based on the fact that both the model structure and model coefficients are determined only from the properties of the data observed. The model is made according to the principle of a successive sophistication at the transfer from simple one-factor dependences under homogeneous conditions to multi-factor and composite structures. Finally, the empirical analysis results in the determined laws of informative factors in time and, depending on the obtained results, a deterministic part of the model is formed (if factors depend on time) or statistic one (if no dependences on time are established). During the final stage, all the parts of the model with the account of informative indices interaction, are combined into one model. The example of such way is in the article [5].

6.6 The prediction is possible if: there is a dependens of cycle characteristics upon time or the process is compositional (with different scales). If even the first requirement is not met, mean trends may be applied to predict large-scale components of the compositional process, which also makes the general uncertainty lower.The main postulate of this approach is: a large-scale process (or processes) originating today, cannot be over tomorrow. The main criterion of possibility and efficiency of forecasting is the correct verification of forecasts results [12].

6.7 Probability or return period (pt) in hydrological computations is interpreted as period of water project. For traditional way, hydrological computations in changing natural conditions are based on the following equation:

$$Ypt = \sum_{i=1}^{k} f(E(Pi)) + Ycl / t=pt + Yhu / t=tp , \qquad (12)$$

where f(E(Pi)) - a composition of distributions of k stoshastic components;

Ycl - forecast of runoff component, connected with climate change on period pt-years in future;

Yhu - forecast of runoff component, connected with human activaty on period pt-years in future;

Ypt - computed hydrological value for pt-return period.

For new approach a probability or return period can be obtained by different situations, for example, a synchronous appearance of maximum amplitudes in all different-scale components. The probability of such situation (return period) for random and undependens characteristics of different components is:

$$P = 1/KA1 * 1/KA2 * \ldots 1/KAk, \qquad (13)$$

where KA1, KA2, ..., KAk - numbers of amplitudes in every homogeneous componens from 1 to k,

1/KA1, ... 1/KAk - probabilities of maximum amplitudes for every quasi-homogeneous components.

7. CASE STUDIES

7.1 Climate Characteristics Analysis.

A) 15-year series of mean daily air temperature have been considered for 6 stations, i.e. Moscow, Verebie, Sortavala, Vologda, Tallinn and St.Petersburg, for 1966-1980. The empirical methods, proposed in this paper,were used to separate a sophisticated process into homogeneous components; synptic factors, generalized for a 10-day period, such as extrema characterizing frequency of air masses intrusion, extrema and their differences in warm and cool masses, and anomality factor determining deviation from average conditions, were determined. The analysis shows that all these characteristics have an evident regular component during a year, or annual variation. Therefore, a long-term series of the coefficients of the function of annual variation of the selected synoptic characteristics for all these stations was investigated to evaluate the climate variability; this made it possible to conclude that there were no laws and there is a stability of climate in this territory for 15-year period.

B) A similar analysis of long-term variations of air masses has been made for stations along the circles of 56 NL and 64 NL to evaluate the dynamics of the cold coming from the pole relative to the rest of the Northern Hemisphere territory. The north pole, as defined in [3], for example, is the basic part of the climate system, which determines and identifies the total climate variation on the planet. Investigations of the laws of cold air masses temperature over the territory make it possible to determine the "cold" area which has a pulsating orientation from north-east to south-west in Eastern Siberia. The analysis of climate characteristics in individual stations for 1936-1980 shows a rise of minimum temperatures from the middle of 1950s which corresponds to general conclusions made in [1], at the background of which pulsations of quasi-there-year cycles with the cycle periods from 2 to 5 years are observed.

C) Interaction between different climate characteristics has been analysed. The basic idea was, that various types of air masses were characterized by different moisture content. For Syktyvkar station during 1966-1980 air masses were separated into cool and warm, and the total precipitation was computed inside cool and warm masses individually. It appeared, that the moisture content in the warm masses coming from the North and Central Atlantic Ocean was greater than in the cool masses coming from the Polar Ocean. Relations between temperature characteristics and moisture content are different in various air masses and they have the extremum related to baric formations of the spring-autumn period during the maximum temperature gradient between land and ocean.

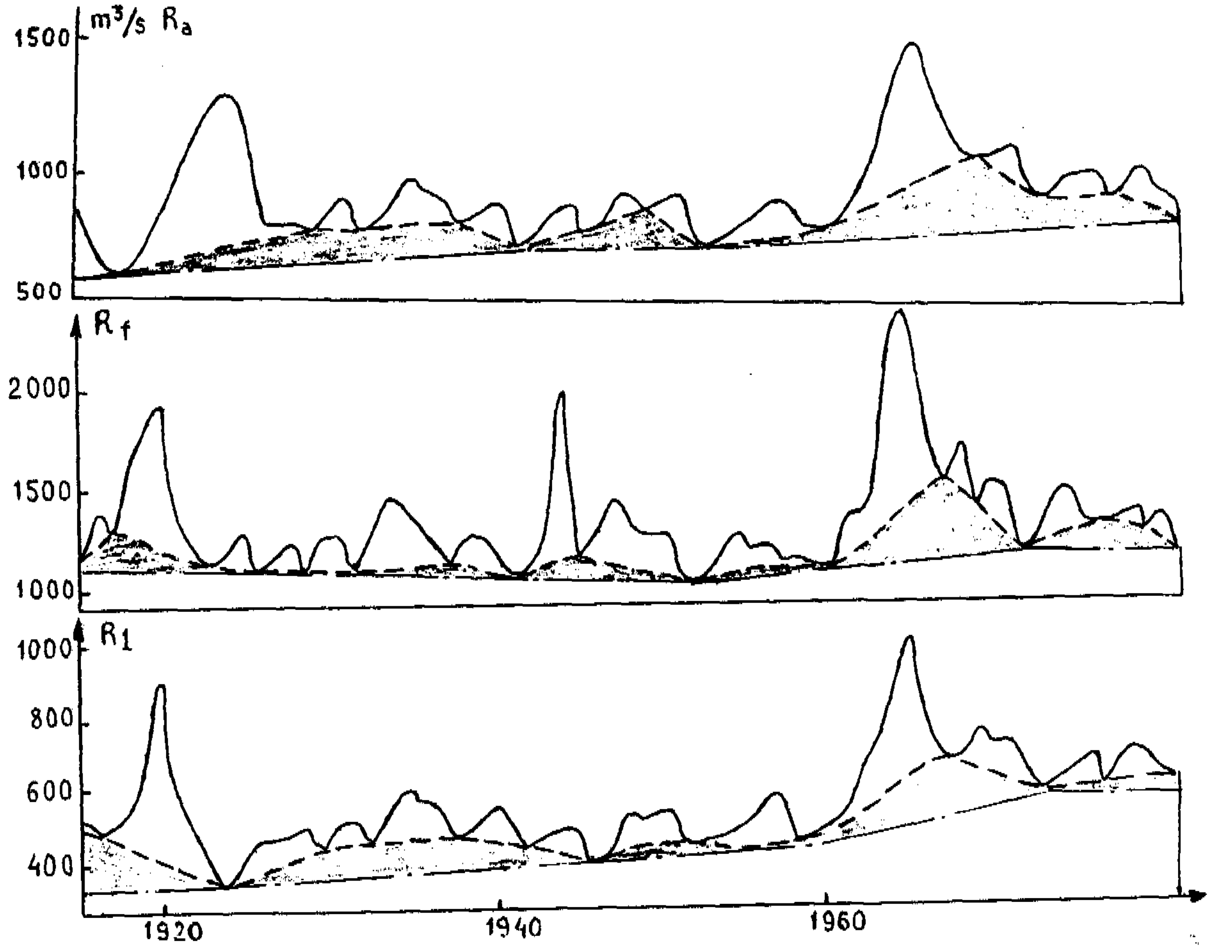
7.2 Runoff Characteristics Analysis.

A) Long-term series of water discharges have been considered for some European rivers of the territory of Russia. The proposed methods have been applied to separate the sophisticated process into quasi-homogeneouse components, the number of which equals three for the majority of cases, and four in some cases only. Mean period of cycles of the least-scale component varies from 3 to 4 years, that of the next scale - from 10 to 12 years. Large-scale components are presented by the period of several dozens of years (40-70 years). Secular variations which may be connected with the global climate changes are displayed as a part of the cycle of the maximum-scale process. The trend of the secular-scale proces over territory is extremely variable: it ranges from positive to negative and in some cases it is not observed at all. A forecast of characteristics of the process cycles of the first and second scales for the last 23 years has been made for the longest series of the Neman river at Smalininkai. The error of the maximum value from the synthesized prognostic series was 4.7%, minimum one - 12.9% and the mean value - 3.6%.

B) Long-term series have been considered for two sites on the Nile river: Nile - Aswan Dam (Egypt, from 1869) and White Nile - Malakal (Sudan, from 1912) for annual runoff, monthly flood run-off and low flow.The truncation method was used for decomposition and long-time analysis of those hydrological characteristics. It is known that the Large Aswan Dam was built in 60th and analysis of long-time fluctuations was executed before this date. As it has been determined, all natural time series of hydrological characteristics consist of two different scale constituents and part of the third process. Mean period of cycles of the least-scale component is 4 years (from 2 to 12). Period of cycles of next scale (middle scale process) has mean value 25 years for annual runoff (with variation from 12 to 30) and mean is 16 years (from 8 to 28) for maximum and low-flow time series. The third process of large scale takes place as a small positive trend in all time series and more significant for upper site. Example of decomposition of annual (Ra), flood (Rf) and low flow runoff (Rl) for Malakal site is shown on Fig.3.

Some regular properties (cyclic laws and positive trend) were determined for periods and amplitudes of the small-scale process at the Malakal site too. Those laws could be used for demands management of Aswan Dam in the future.

Figure 3. Decomposition of runoff time series at Malakal site



7.3 Spatial Modeling

Two case studies of spatial modeling are considered for application of proposed poli-linear decompositional method. In first case historical time series of monthly mean surface temperature from 1891 to 1990 and series of 500 mb Heights (geopotential heights) from 1949 to 1990 are used for modeling. Those data were obtained from time series of observations as well as from the remote sensing lately and were interpolated into units of regular grid with step 10" along the longitude and 5" along the latitude. It has been established, that parameters At and Bt for temperature have a century-long trend and correlation coefficient between them R=-0.81. The standard deviation of the process of macrosynoptical scale St has a step-trend with sharp fall at the beginning of 60th. The similar results has been obtained for parameters of H=500mb - Ah, Bh and Sh. Models of intercommunication between coefficients of separations for temperature and geopotential heights are built on the base of following common equation:

$$F\ (At,Bt,St,Ah,Bh,Sh) = 0 \tag{14}$$

and for particular case, for example At:

$$At=g(Bt,St,Ah,Bh,Sh) + E \tag{15}$$

The results of modeling are shown in Table 1.

Table 1 Poli-linear regression models for common characteristics of fields of temperature (T) and H500

Function	Factors	Coefficients	Explained part of variance (in % to common)	Correlation coefficient
At	Bt	-0.1495	44.1	0.906
	Bh	0.2122	17.1	
	Ah	0.1060	10.0	
	Ah*Bh	-0.2274	10.8	
Ah	At	0.4828	37.0	0.879
	Bh	-0.7557	21.9	
	At*St	-1.1038	10.2	
	Bt*Bh*St*Sh	-3.4308	8.1	
Bt	At	-0.0109	46.4	0.906
	St	-0.0104	20.7	
	At*Bh	0.0161	14.9	
Bh	Bt	0.0166	19.4	0.771
	Bt*Ah	0.0316	15.1	
	At	0.0193	11.8	
	Ah	-0.0061	8.0	
	At*Ah*Bt	-0.0297	5.1	
St	Sh	0.4367	59.6	0.909
	Bt	-0.5154	11.1	
	Bt*Sh	-0.8087	8.5	
	At*Bt*Ah*Bh	-2.3587	3.5	
Sh	St	7.2220	60.0	0.792
	At*Bt*Ah*Bh	-10.810	2.8	

In the second case the regional model of water cycle is obtained for basin of Upper Volga in Russia by a poli-linear decomposition method. For this aim time series of main water cycle components: monthly mean temperature, runoff, precipitation are separated and coefficients of separation were analysed in time and were connected one with others for building of spatial models. The results of intercommunication between coefficients of separation for different hydrometeorological characteristics are shown in Table 2.

Table 2. Poli-linear regression models for common characteristics of fields of runoff, precipitation and temperature in Upper Volga region

Function	Factors	Coefficients	Explained part of variance	Correlation coefficient
Ar	Br	1.980	76.1	0.791
	Sr	0.364	19.6	
	Ap	0.038	4.3	
Br	Ar	0.181	100.0	0.701
Sr	Ar	0.604	91.6	0.612
	Sp	-0.074	8.4	
Ap	Sp	0.624	44.7	0.694
	Ar	1.060	23.6	
	Bt	-34.45	11.8	
	At	-2.850	19.9	
Bp	Ap	0.024	100.0	0.340
Sp	Ap	0.266	83.5	0.507
	Sr	-0.502	16.5	
At	Bt	12.493	27.9	0.899
	St	1.000	72.1	
Bt	At	0.080	27.9	0.899
	St	-0.080	72.1	
St	At	1.000	18.9	0.899
	Bt	-12.49	81.1	

7.4 Analysis of Global and External Factors of the Climate System.

Series of mean air temperature in the Northern Hemisphere, radius-vector of the pole of the Earth's rotation and solar radiation (Wolf numbers) have been considered as factors.It has been established that the global air temperature and radius-vector contain three quasi-homogeneous components while solar radiation.

contains two such components. The factors of solar radiation cycles and those of radius-vector are most stable for forecasting purposes. The global air temperature, according to the greatest component, tends to a rise, and the solar radiation will be at the recession phase of the secular cycle at the end of the XXth century and at the beginning of the XXIst century.

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A REAL-TIME STOCHASTIC DYNAMIC PROGRAMMING MODEL FOR MULTI-PURPOSE RESERVOIR OPERATION

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ABSTRACT

Dynamic programming (DP) models have been used to solve multi-purpose reservoir management and operation problems. The problems are often complicated due to the conflicts existing among various reservoir purposes including satisfying agricultural, industrial and municipal demands, generating hydro-power, improving river transportation and navigation, recreation,..etc. The uncertainty associated with reservoir inflows adds greatly to the complexity of the problems.

A decision support package is developed to help decision makers and reservoir operators to derive optimal reservoir operating policies and to investigate the importance of considering inflow forecasts into the optimization procedure. The decision support package includes four module to derive these optimal policies and test them.

The first module is an inflow statistical analysis model to analyze the inflow data and calculate inflow probabilities; either independent probabilities or conditional probabilities. The second module is a stochastic dynamic programming model (SDP) developed to derive monthly optimal operating policies for single multi-purpose reservoirs. Three formulations of its recursive equation are used to examine and evaluate the importance of including the correct inflow variable either historical inflows or inflow forecasts into the optimization scheme. The third module is an auto-regressive forecast model to determine monthly inflow forecasts into the reservoir to be used in one of the recursive equation formulations. The forecast model uses streamflow records at various upstream locations on the Nile tributaries during previous months. The fourth module is a monthly reservoir simulation model to examine and validate policies obtained from the optimization model.

The package is applied to the case study of the High Aswan Dam (HAD) in Egypt. The dam is the major regulatory project on the River Nile and it controls releases from Lake Nasser reservoir to satisfy the country's agricultural, industrial and municipal demands. The package is used to obtain reservoir operating policies. Policies obtained are simulated over the entire period of operation of the HAD considering severe drought conditions.

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1. INTRODUCTION

Egypt's population rate is increasing very rapidly. The country is challenged with the great need for food and fibre production under constraints of limited water resources. The Nile River is the main source of water to the country (Fig. 1) providing it with 95% of its water requirements for the various uses. Egypt has a fixed share of the Nile water according to the 1959 agreement with Sudan. The High Aswan Dam (HAD) is the major regulatory facility on the Nile controlling releases from Lake Nasser, the impoundment upstream the reservoir. The High Aswan Dam was designed mainly to satisfy irrigation requirement according to a monthly demand schedule. The dam also serves in protecting the country against high floods, which in the past used to threaten cities and villages each year during the flood season. The power facility at the dam site used to generate about 80% of the country power requirements. The problem of defining optimal release rules for the HAD is complicated by the multiplicity of these conflicting objectives, along with the uncertainty associated with the coming river flows into the reservoir lake. It is not expected within the near future that this share would be increased, unless one of the Upper Nile projects would be implemented and be operational, and that is why the decision makers in the Ministry of Public Works and Water Resources (MPWWR) are trying to investigate opportunities for rationalizing and optimizing the use of the Nile water through the use of mathematical models determining the optimal reservoir releases.

2. STOCHASTIC DYNAMIC PROGRAMMING

Management and operation of multi-purpose reservoir systems is a difficult problem facing decision makers and reservoir operators. The decision process in the operation of any multi-purpose reservoir is often complicated by conflicts existing among various purposes which the reservoir system is designed to satisfy. Dynamic programming (DP) models have been used in many previous studies to determine optimal reservoir release policies and to solve reservoir operation problems. Dynamic programming models can be easily formulated to solve deterministic or stochastic operation problems. Stochastic DP models have proven to be a practical tool to solve these problems (Alarcon and Marks, 1979; Bras et al., 1981; Stedinger et al., 1984; Kuo et al., 1990; Huang et al., 1991). Georgakakos (1989) investigated the importance of including inflow forecast information into the formulation of the optimizing technique and concluded that policies obtained based on updated inflow forecast information are better than polices obtained from conventional stochastic DP using the lag-one Markov inflow models.

It is intended through this study to show the importance of including inflow forecasts and their statistics into the optimization scheme to obtain reservoir operation policies. A real-time stochastic dynamic programming (DP) model is developed to solve the problem of deriving optimal reservoir operating policies based on actual inflow records along with inflow forecasts. A statistical analysis program is implemented to produce inflow ranges and associated probabilities required by the stochastic DP to derive the optimal policies. An auto-regressive forecast model is developed to provide inflow forecasts that can be directly included in the stochastic DP formulation. The stochastic DP and the statistical analysis program are incorporated into an integrated decision support system (DSS).

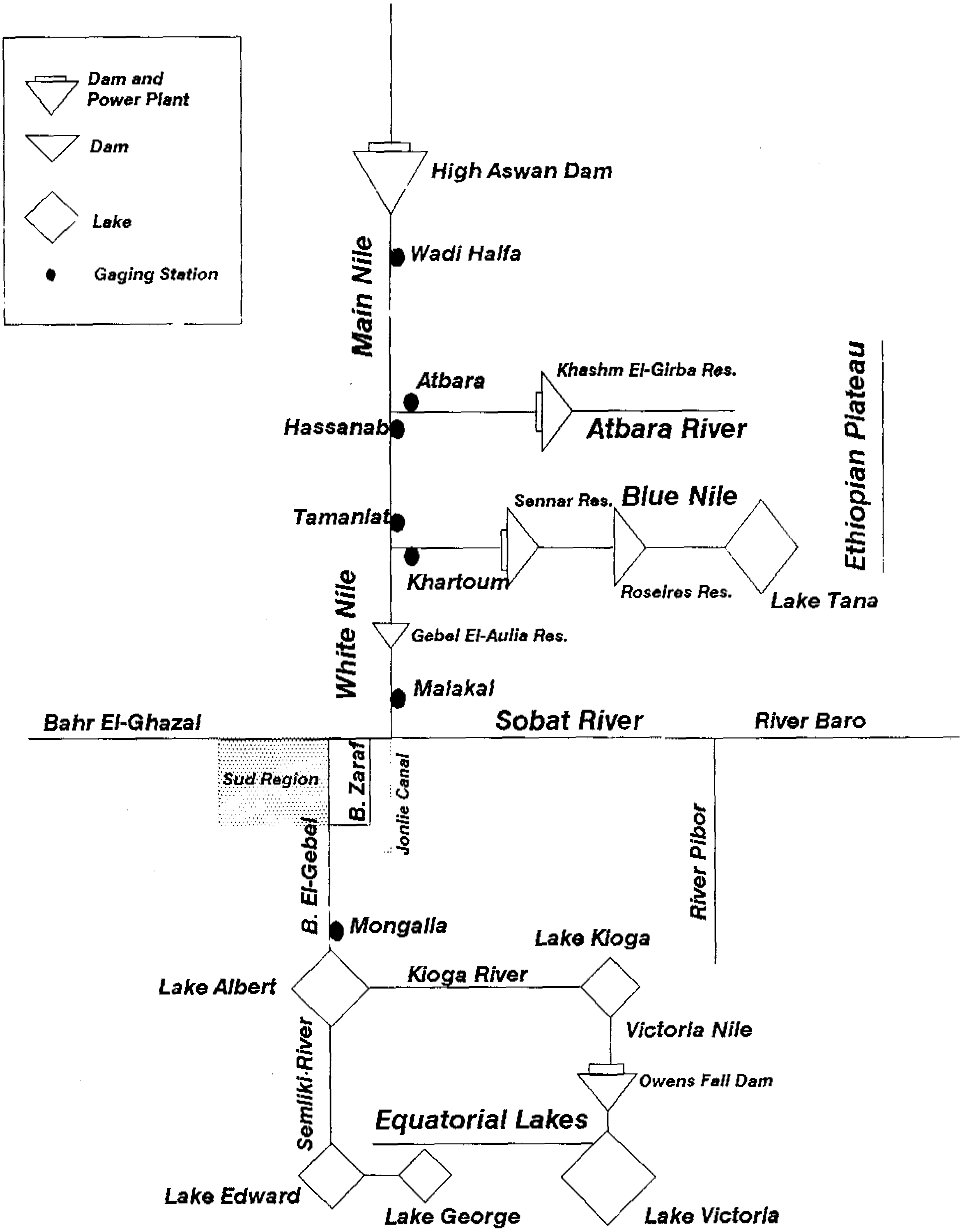

Fig. 1 Schematic Representation of the River Nile Basin

Three DP formulations are used to derive optimal reservoir operating policies using different recursive equation formulations considering three different monthly reservoir inflow assumptions. The three recursive equation formulations are based on consideration of uncorrelated historical inflows, consideration of lag-one or first order Markovian inflows, or consideration of updated inflow forecasts. The problem is solved using the backward solution of the stochastic DP over a finite time horizon determined after the steady state conditions are satisfied and constant operating policies are reached over all discrete values of the state variables. The boundary conditions at the terminal stages for each formulation are provided for the solution of that formulation. The recursive equation in all cases is formulated as an optimal value function. Assumptions associated with the solution of each formulation are presented in the following sections and the problem is subject to various reservoir physical and operational constraints. The mass balance at the reservoir site (the reservoir state equation) can be applied using either the inverted or the non-inverted form. Policies obtained after solving the three types of recursive equations are examined and compared using a reservoir simulation model.

2.1. Case of Uncorrelated Inflows

The first recursive equation formulation assumes uncorrelated monthly inflow series which are discretized into a number of discrete representative monthly inflow values. Unconditional (independent) probabilities for the different monthly discrete representative inflows are calculated. The representative monthly inflows and their unconditional probabilities are employed into the stochastic DP formulation. The recursive equation in this case is written as:

$$F_t^n\ (S_t) = \underset{R_t}{Min} \sum_{Q_t} P_{Q_t}\ [\ C(S_t,\ R_t)\ +\ \beta\ F_{t+1}^{n-1}\ (S_{t+1})\ ]$$

where $F_t^n(S_t)$ is the minimum expected value of the system performance from the current period t till the end of the time horizon T when the discrete reservoir storage at the beginning of time period t is S_t, P_{Q_t} is the unconditional probability for the discrete representative inflow value Q_t to occur during period t, $C(S_t,\ R_t)$ is the reservoir operational cost function, R_t is the release decision taken for period t, β is the discount factor as $\beta = \frac{1}{1+r}$ where r is the interest rate (r=0 for the undisc·unted case), and n is the number of periods calculated backward from the end of the time hori.on " ie current period t.

This recursive equation is subject to either the non-inverted (stantard) or to the inverted state equation described as follows:

$$S_{t+1} = S_t + Q_t - R_t - L_t(S_t, S_{t+1})$$

or

$$R_t = S_t - S_{t+1} + Q_t - L_t(S_t, S_{t+1})$$

where S_t, S_{t+1}, R_t and Q_t are defined as before and $L_t(S_t, S_{t+1})$ is the loss term including various losses calculated at the reservoir site during time period t as a function of initial and final reservoir storage volumes. The calculation of the loss term correctly in the first case requires a number of iterations assuming values for the random state variable S_{t+1} as it appears on the right-hand-side of the equation. The losses are calculated based on the assumed S_{t+1} and then verified by re-calculating a new S_{t+1} using the calculated losses. The losses in the second equation are calculated directly without iteration as they are dependent on two known variables.

Both state equations are subject to the following constraints on both the state and decision variables:

$$S_{\min} < S_{t+1} < S_{\max}$$

and

$$R_{tmin} < R_t < R_{tmax}$$

where S_{min} is the reservoir minimum storage volume (dead storage zone for sediment trap), S_{max} is the maximum reservoir storage volume (reservoir capacity), R_{tmin} is the minimum allowable release during time t, and R_{tmax} is the maximum allowable release during time t.

Optimal release policies obtained would be functions of the reservoir storage at the beginning of the current month. The release policies obtained from this formulation will be examined through the simulation model and compared with policies derived based on the other inflow assumptions.

2.2. Case of Lag-One Markov Correlated Inflows

In this case the statistics of the monthly reservoir inflows are calculated. If the correlation between successive monthly inflows is determined to be high, this second formulation is used. The monthly historical inflow series are discretized into a number of inflow ranges with the median of each range employed as the discrete representative inflow for that range. The conditional (transition) probabilities between discrete representative inflows of successive month are calculated utilizing the calculated correlation coefficients. Release policies derived in this case will be functions of the reservoir storage at the beginning of current period and reservoir inflow during the previous period.

The stochastic DP recursive equation is solved using either the inverted or the non-inverted state equation. The recursive equation is expressed as:

$$F_t^n\ (S_t,\ Q_{t-1}) = \underset{R_t}{Min} \sum_{Q_t} P_{Q_t/Q_{t-1}}\ [\ C(S_t,\ R_t) + \beta\ F_{t+1}^{n-1}\ (S_{t+1},\ Q_t)]$$

where $F_t^n(S_t,Q_{t-1})$ is the minimum expected value of the system performance from the current time period t till the end of the finite operational horizon T for the discrete state variables S_t and Q_{t-1}, S_t is the discrete initial reservoir storage, Q_{t-1} is the discrete representative inflow for the previous period t-1, $P_{Q_t/Q_{t-1}}$ is the steady state transition probability of the current period discrete representative inflow Q_t conditioned on the previous period inflow Q_{t-1}, $C(S_t,\ Q_{t-1})$ is the operational cost function resulting from taking decision R_t during time period t while starting from an initial storage S_t, and β is the discount factor. The end of the finite horizon in this case is determined by the time when the steady state conditions are reached and the system performance value and the optimal policies obtained become constants.

This stochastic DP recursive equation is solved using the following boundary condition at the terminal time period T:

$$F_T^0\ (S_T,\ Q_{T-1}) = G$$

where G is the given return function value at the end of the last stage of the required terminal system conditions. The recursive equation in this case is also subject to the mass balance constraint at the reservoir site. The equations for the upper and lower bound on the state and decision variables also apply. The stochastic DP formulation would be solved over a number of stages until the expected change in the total system performance $F_{t-M}^{n+M} - F_t^n$ is constant over all discrete values for S_t and Q_{t-1} for all time periods within a year where M is the total number of periods within a year assuming an interest rate of zero. At this stage, the policy is considered to have reached the steady state. Policies obtained from solving this stochastic DP formulation are considered stationary policies as the cost function and inflow statistical characteristics do not change with time.

2.3. Case of Considering Inflow Forecasts

For this third stochastic DP formulation, it is assumed that an available reservoir inflow forecast model is capable of producing the monthly inflow forecast with a great reliability . This forecast model can be either physically based or statistical model. Both the recorded inflows and the inflow forecasts will be used in the optimization procedure. The recursive equation in this case can be written as:

$$F_t^n\ (S_t,\ \hat{Q}_t)\ =\ \underset{R_t}{Min} \sum_{Q_t} P_{Q_t/\hat{Q}_t}\ [C(S_t,\ R_t)\ +\ \sum_{\hat{Q}_{t+1}} P_{\hat{Q}_{t+1}/Q_t}\ F_{t+1}^{n-1}\ (S_{t+1},\ \hat{Q}_{t+1})$$

where $F_t^n(S_t,\hat{Q}_t)$ is the minimum expected value of the system performance calculated from the current time period t till the end of the operational horizon when S_t is the beginning of month discrete storage and $\hat{Q}_t$ is the current period representative inflow forecast, $P_{Q_t/\hat{Q}_t}$ is the probability of the current period's discrete inflow conditioned on current period's inflow forecast, $P_{\hat{Q}_{t+1}/Q_t}$ is the probability of next period's discrete inflow forecast conditioned on current period's inflow. All other variables are defined as in the previous cases.

Policies derived for this case are considered to be non-stationary policies because the inflow statistical characteristics (transition probabilities) change from one year to another. To obtain these non-stationary policies from a backward solution of this recursive equation, a boundary condition should be provided for the terminal stage of a finite time horizon. This boundary condition is the value of the optimal return function obtained from the solution of the steady state case considering lag-one (first order Markovian) inflows for all combinations of discrete reservoir storage volumes and discrete reservoir inflows.

For this formulation of the recursive equation, a streamflow forecast model is required to provide the inflow forecasts used in the formulation. This forecast model can be either physically based or statistical model depending on the information available for the river system under consideration. After the inflow forecasted series are obtained, the forecasted series are discretized into a number of representative inflows and the probabilities of the historical recorded inflows conditioned on the forecasted inflows and the probabilities of the forecasted inflows conditioned on the recorded inflows are determined and employed into the recursive equation formulation.

3. APPLICATION OF THE STOCHASTIC DP MODELS

The three models are applied to the case study of the High Aswan Dam considering an objective function that minimizes the squared deviation from the monthly irrigation targets specified by the Ministry of Public Works and Water Resources (MPWWR). Monthly irrigation targets used are the monthly values for water required for the monthly cropping pattern provided by the Ministry of Agriculture and Land Reclamation (MALR) in Egypt considering conveyance and application losses in the irrigation system. Monthly energy targets are the maximum values of energy that can be generated for the optimal reservoir conditions. The constraints on this objective function are the mass balance equation at the reservoir site, minimum and maximum allowable monthly releases, and minimum and maximum reservoir storage volumes. The reservoir mass balance equation includes various types of losses as they are calculated using the reservoir initial storage volume. The state equation is suggested to be used in the non-inverted form to obtain monthly reservoir release policies as function of discrete reservoir storages and discrete inflows. All equations used to calculate reservoir losses and all constants used in these equations are obtained from the Planning Sector of the MPWWR in Egypt.

The minimum value (lower bound) of the reservoir storage volume is considered to be 37.2 billion cubic meters (BCM) which represents the reservoir dead storage to entrap sediment. The maximum value (upper bound) of reservoir storage is considered to be 168.9 BCM representing the reservoir storage capacity. The minimum and maximum allowable releases are suggested to be 15.0 and 0.0 BCM. If the release in any month exceeds the value of 7.6 BCM, which is determined to be the maximum release that would not cause degradation in the river channel downstream the dam, a high penalty term is added to the objective function based on the release deviation from the 7.6 BCM value. The reservoir inflows used are the natural streamflows at Aswan available for the period 1912-1989. From this data, discrete representative inflow values, conditional and unconditional probabilities of these inflows will be determined.

3.1. The Statistical Analysis Program

The statistical analysis is required by the stochastic DP to identify the inflow probability distributions and to calculate the corresponding transition probability matrices. The inflow basic statistics are calculated based on the available historical inflow records. Correlation coefficients between reservoir inflows of any two consecutive months are determined and used to calculate inflow transition (conditional) probabilities. If the correlation coefficients are of small values, inflows are uncorrelated, the inflow records are sorted and divided into a number of inflow classes based on the user's choice. The inflow median of each class is chosen to represent the class and the probability of each class is calculated to be used in the stochastic DP formulation for the case of the uncorrelated inflows.

If the inflows are determined to be serially correlated, a first-order Markov chain is assumed. The probability density function to fit the continuous monthly inflow record is identified. The inflows are divided into a number of representative inflow ranges, specified by user, and the median of each range is defined. Transition (conditional) probabilities between flows of consecutive months are calculated based on the values of the monthly inflow statistics. Inflow medians and conditional probabilities are used in the steady state case of the stochastic DP formulation.

3.2. CSUDP Model

The DP code used is the CSUDP model which is a generalized dynamic programming code that has been used by researchers as a tool to derive optimal operating policies in reservoir management problems. The program is developed by Dr. John W. Labadie, Professor of Civil Engineering at Colorado State University. The CSUDP solves minimization and maximization objective functions in a backward solution mode. It is capable of solving deterministic and stochastic problems. The program can solve stochastic problems where inflows are considered random discrete variables with known probabilities, that can be either independent (unconditional) or conditional (transition) probabilities. The CSUDP computes not only optimal set of open-loop policies but also a set of closed-loop policies conditioned on various discretizations of the state variables. The program has the facility which allows the user to create and edit the control data easily through a menu driven option. It also allows the user to write the subroutine which describe the problem.

The user of the CSUDP has to prepare three FORTRAN subroutines to describe the system under consideration and to compute the objective function values. The three subroutines include a STATE subroutine which mainly describes the mass balance equation at the reservoir site defined as the state transformation equation, an OBJECT subroutine that calculates the objective function values, and a READIN subroutine that reads additional data required to calculate the state equation and the objective function.

The constraints on values of state and decision variables are introduced as upper and lower bounds. The CSUDP allows the user to edit the discretization of state and decision variables and the optimization procedure is performed over all discretizations of either state or decision variables.

The state equation of the CSUDP is formulated using the non-inverted form as follows:

$$S_{t+1} = S_t + Q_t - R_t - SD_t - E_t - SP_t - SB_t - TS_t$$

where S_{t+1} is the reservoir storage at the end of time period t, S_t is the reservoir storage at the beginning of time period t, Q_t is the reservoir inflow during period t, R_t is the release during time t, SD_t is the monthly Sudan abstractions, E_t is the evaporation losses during month t, SP_t is the seepage losses for time period t, BS_t is the bank storage for month t, and TS_t is the Toshka spills in case of reservoir elevations greater than 178 m. during any month. All losses are calculated as function of reservoir storage volumes at the beginning and at the end of month t.

For the first case of recursive equation formulation, the inflows are assumed to be uncorrelated. Five discrete representative inflows are determined and independent (unconditional) probabilities for these representative inflows are calculated. The problem is solved over a 60-stage (5-year) time horizon after which the steady state policies are reached. The release policies are obtained for all discrete values of reservoir storage state variable. In the second formulation of the recursive equation, the monthly inflows are assumed to be lag-one correlated. Five discrete representative inflows are determined for each month and the conditional probabilities of the inflows on a certain month conditioned on the inflows of he previous month are calculated. Steady state policies are achieved after the calculation of 60 stages. Before presenting the third formulation of the stochastic DP, a brief description of the forecast model used is presented in the following section.

3.3. Forecast Model

For the third stochastic DP formulation, an auto-regressive (AP) forecast model is proposed to be used to forecast monthly reservoir inflows as the AR models are easily developed and usually forecasts obtained from them are sufficiently reliable to be used in solving reservoir operation problems. The AR forecast model is a modified version of a previously developed one for the River Nile flow at Aswan. In this model, the naturalized monthly streamflow series at various River Nile gaging sites are obtained for the period 1912 till 1989 from the Planning Sector in the MPWWR in Egypt. These naturalized flows are used to predict the Lake Nasser reservoir inflows. Only four gaging sites are considered to represent the main tributaries of the River Nile. The travel time between the farthest gaging site and the reservoir is estimated to be six months during low flow season. Correlation coefficients between reservoir inflows and streamflows at various sites are calculated for up to six months lag periods. These coefficients are used to determine which streamflows at which sites should be included in the step-wise regression procedure to derive the forecast equation using MINITAB statistical software. Table 1 presents the values of the regression coefficients and the mean square error obtained from the monthly forecast equations.

Table 1 Regression Coefficients and Mean Square Error of the Forecast Model

Month	Regression Coefficient R^2	Mean Square Error (MSE)
January	90.97	0.052
February	90.72	0.056
March	93.02	0.029
April	87.99	0.102
May	82.11	0.053
June	69.20	0.089
July	32.48	1.388
August	54.00	4.413
September	76.24	2.886
October	71.37	2.544
November	92.61	0.168
December	93.70	0.045

The third formulation of the DP recursive equation required the availability of a reliable forecast model to predict monthly reservoir inflows. The forecast model will be presented in the following section. The forecasted inflow series is discretized and probabilities of inflow forecasts conditioned on recorded inflows and probabilities of historically recorded inflows conditioned on inflow forecasts are calculated. The optimal return function of the steady state solution is used in this formulation at the terminal stage. The model is solved over a 12-period time horizon.

3.4. Lake Nasser Simulation Model

The Lake Nasser Simulation model is used to examine and verify optimal policies obtained from the CSUDP model. The simulation model was developed at the IBM Scientific Center in Italy by a group of Egyptian and Italian Engineers to simulate the response of Lake Nasser under various operating policies considering different sets of inflows. The model is mainly a mass balance equation at the reservoir site. The inflows into the reservoir are the naturalized river inflows calculated at Aswan considering upstream abstracts. The outflow from the reservoir is the release decision obtained from the DP model. The model also calculates various types of losses and computes the amount of energy produced. The output give comprehensive information about changes in reservoir upstream elevations and corresponding storages and amounts of water lost through different loss types. Policies obtained from the three formulations are introduced to the model and compared. The output of the simulation model gives a comprehensive idea about expected reservoir elevations and storage volumes at the beginning of each month, monthly evaporation, seepage and bank storage losses, and monthly energy produced. Three initial reservoir elevations were used to represent an empty reservoir, the average reservoir conditions and a full reservoir.

4. SIMULATION MODEL RESULTS

Fig. 2 displays the simulated reservoir upstream elevations for 8 hydrologic years (drought period) for the three reservoir initial conditions when adopting policies obtained considering uncorrelated monthly reservoir inflows. The figure shows that after 5 years of low flows, the simulated elevations obtained from an initial reservoir elevation of 165 m. are the same as the elevations corresponding to reservoir elevations of 155 m. The elevations simulated considering the high initial elevations are also declining. Fig. 3 displays the monthly shortages that would occur in case of adopting the uncorrelated inflows policies. As expected, the shortages experienced in the case of low initial reservoir elevations are much higher than the shortages in the other two cases. The maximum shortage occurred during the hydrologic year 1984-85 because of the extremely low inflows during that year.

Fig. 4 displays the simulated reservoir upstream elevations when the policies obtained for the steady state case, where the lag-one Markovian inflows are considered. The figure is showing the convergence of elevations during the low flow period. Fig. 5 shows the water shortages anticipated from adopting the steady state policies, while starting from three different reservoir storage levels. The shortages experienced during 1984-85 are the maximum shortages occurring during the simulation period.

Fig. 6 presents the simulated reservoir upstream elevations for both forecast and steady state policies for the years, 1981, 1984, and 1987 considering low initial elevation. The steady state policies yield higher reservoir elevations than the forecast policies. Shortages created from adopting all policies of forecast and steady state cases are demonstrated in Fig. 7. The steady state policies create more shortages than forecast policies in the three simulated years. Policies are also simulated under the assumption of a high initial reservoir storage level. Reservoir upstream elevations and monthly shortages from considering both steady state and forecast cases are exactly the same as shown in Figs. 8 and 9, respectively.

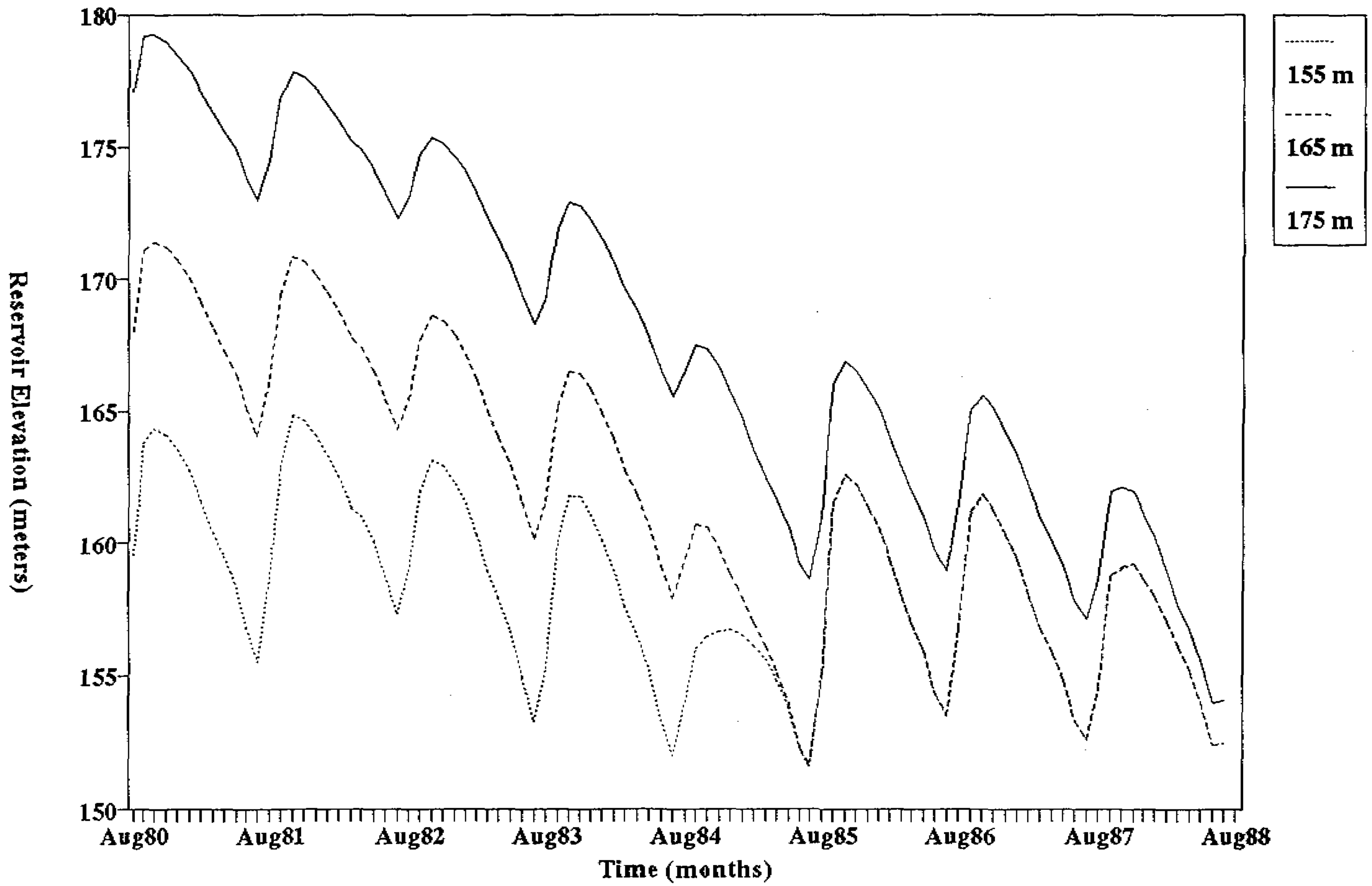

Fig. 2 Simulated Reservoir Elevations for the Case of Uncorrelated Inflows for Three Initial Elevations

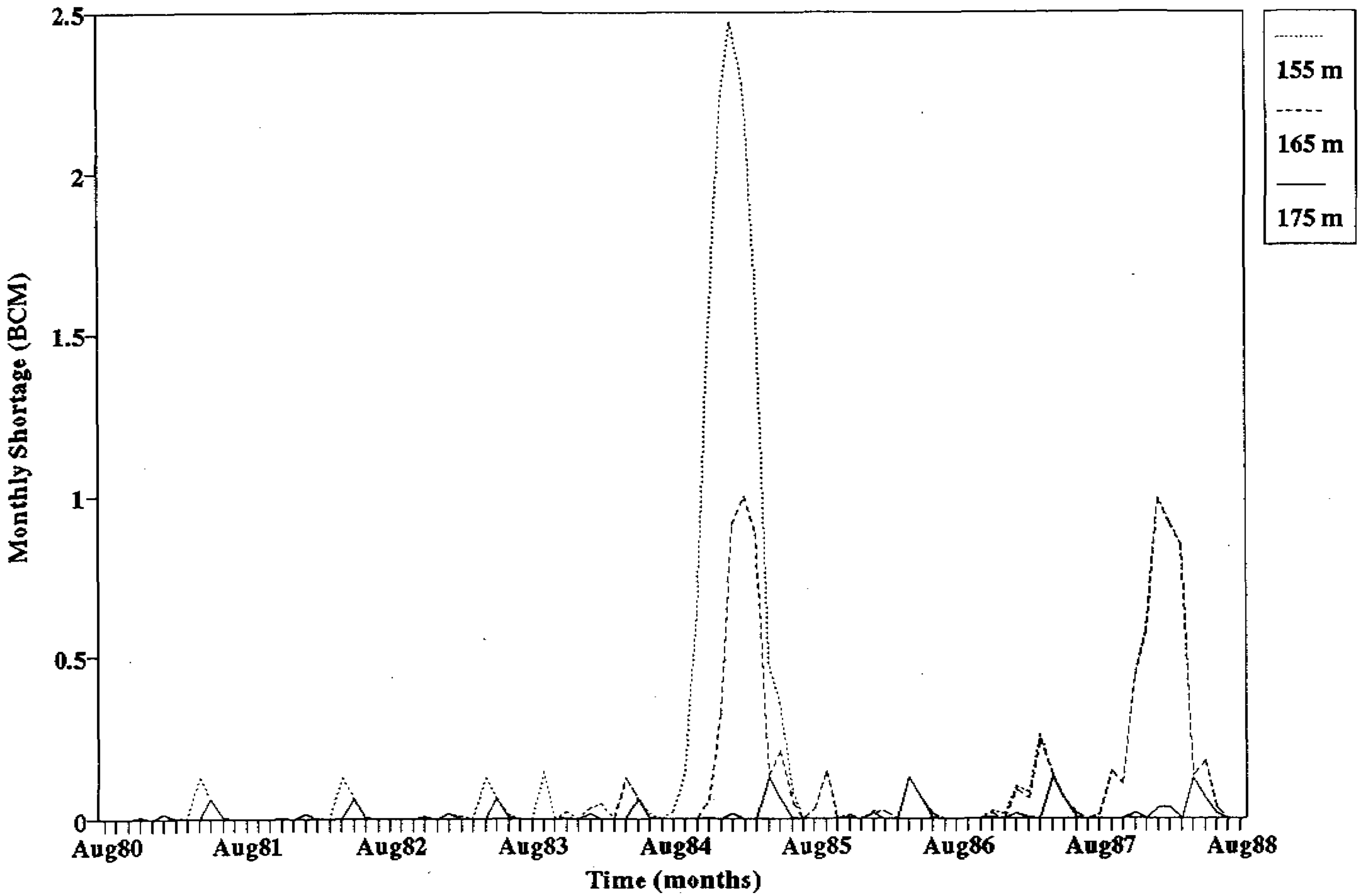

Fig. 3 Simulated Monthly Shortages for the Case of Uncorrelated Inflows for Three Initial Elevations

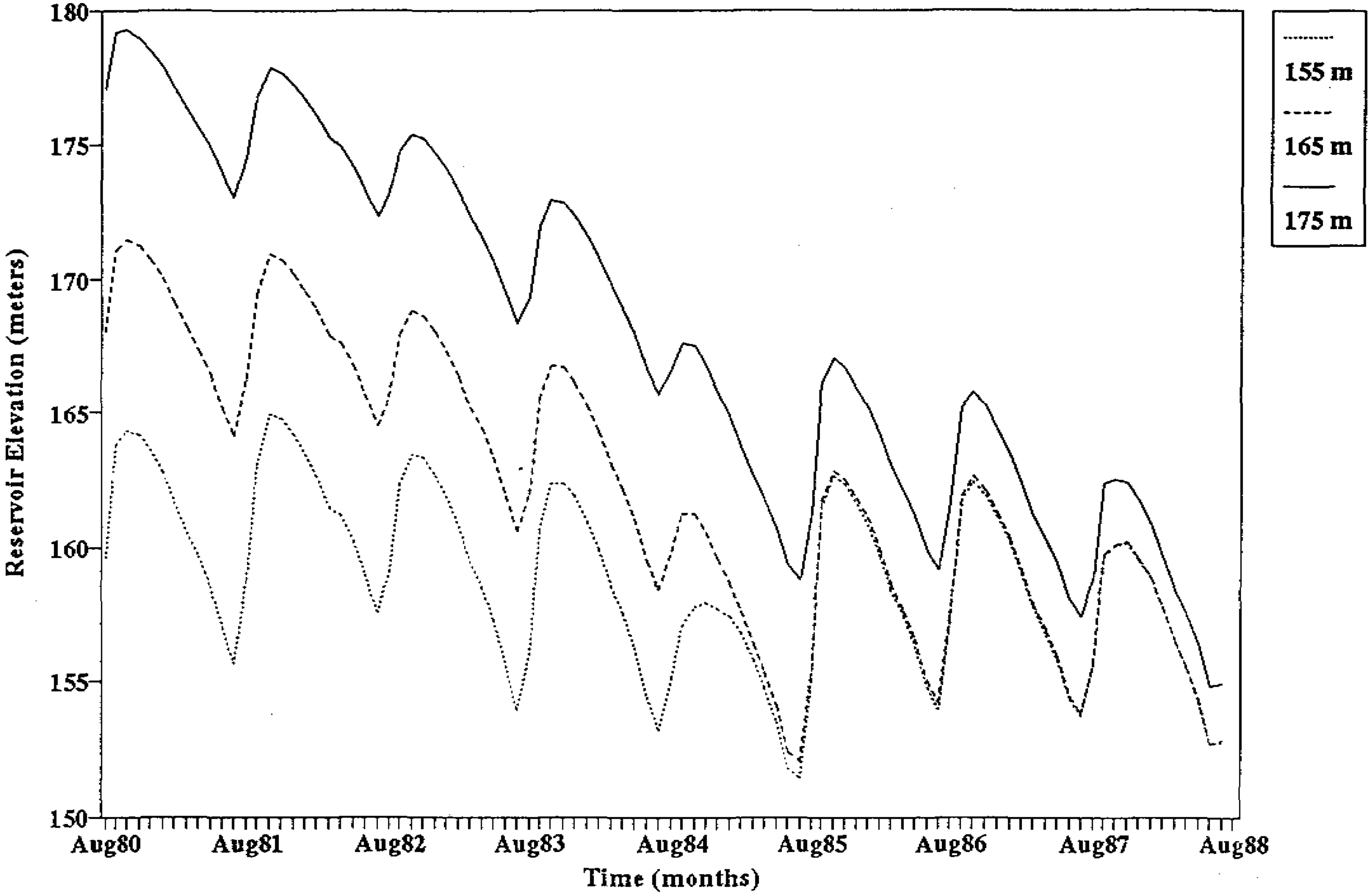

Fig. 4 Simulated Reservoir Elevation for the Steady State DP for Three Initial Elevations

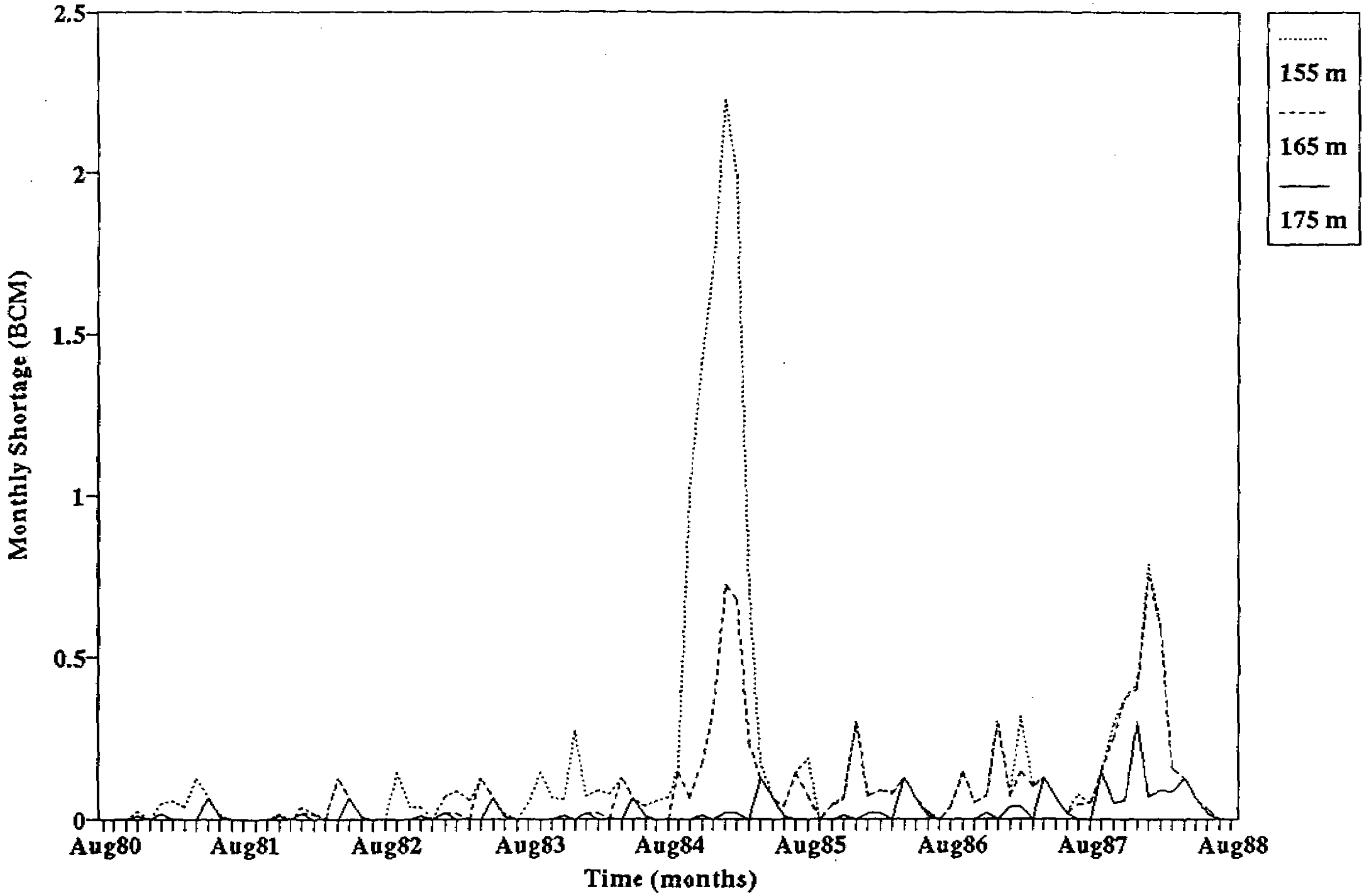

Fig. 5 Simulated Monthly Shortages for the Steady State DP for Three Initial Elevations

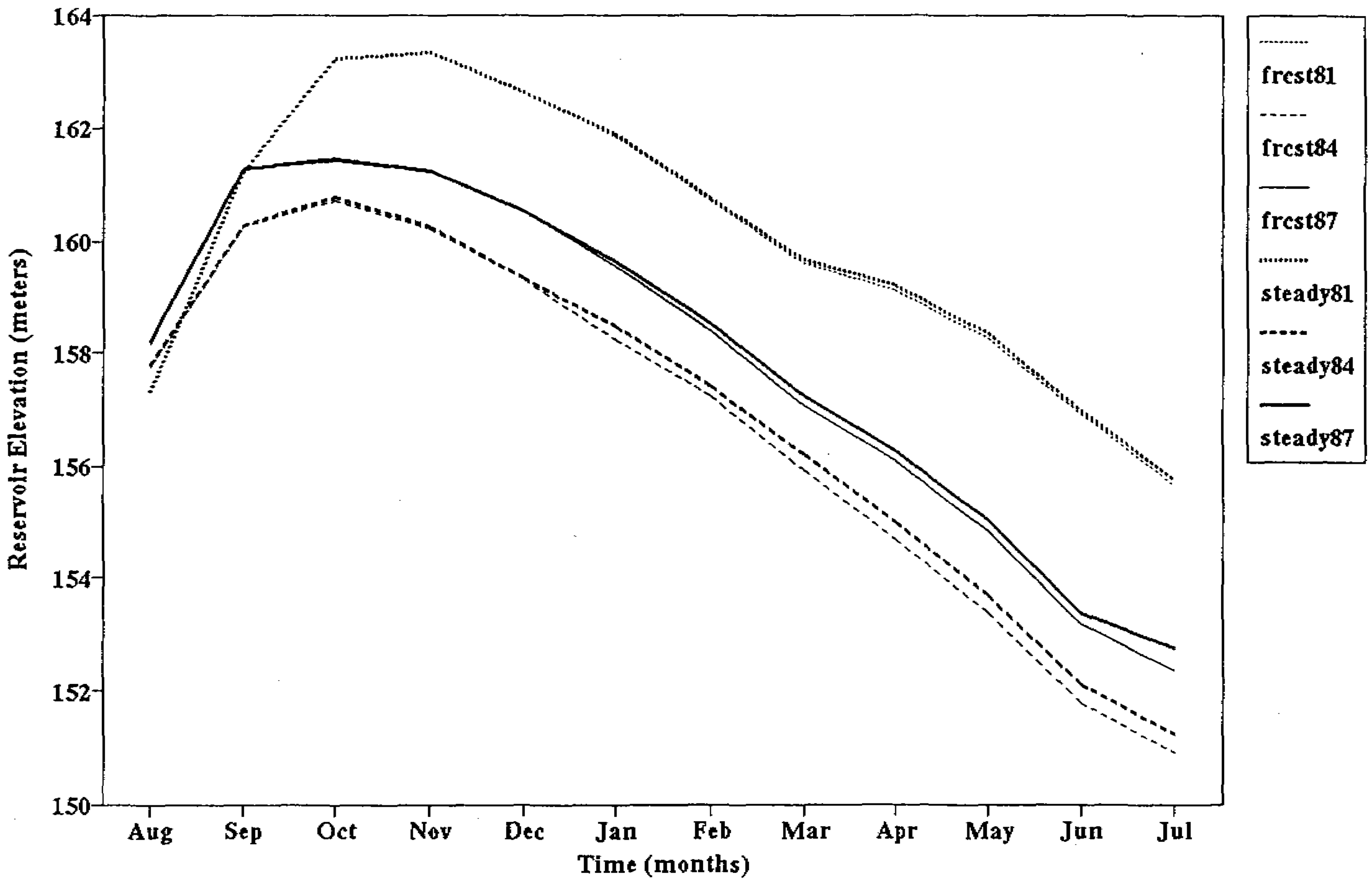

Fig. 6 Simulated Reservoir Elevations for the Forecasted Inflow Series Using Initial Elevation of 155m

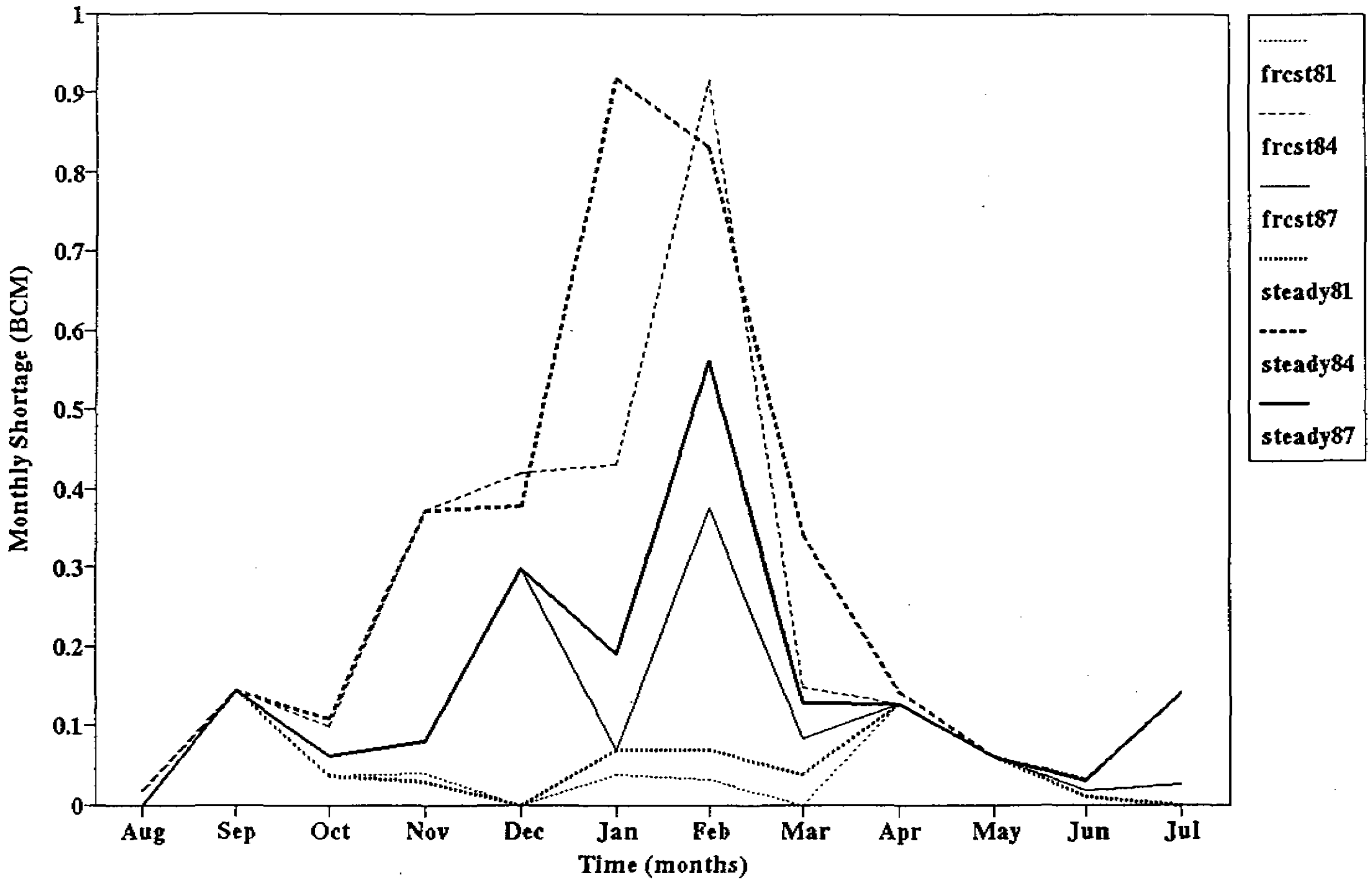

Fig. 7 Simulated Monthly Shortages for the Forecasted Inflow Series Using Initial Elevation of 155m

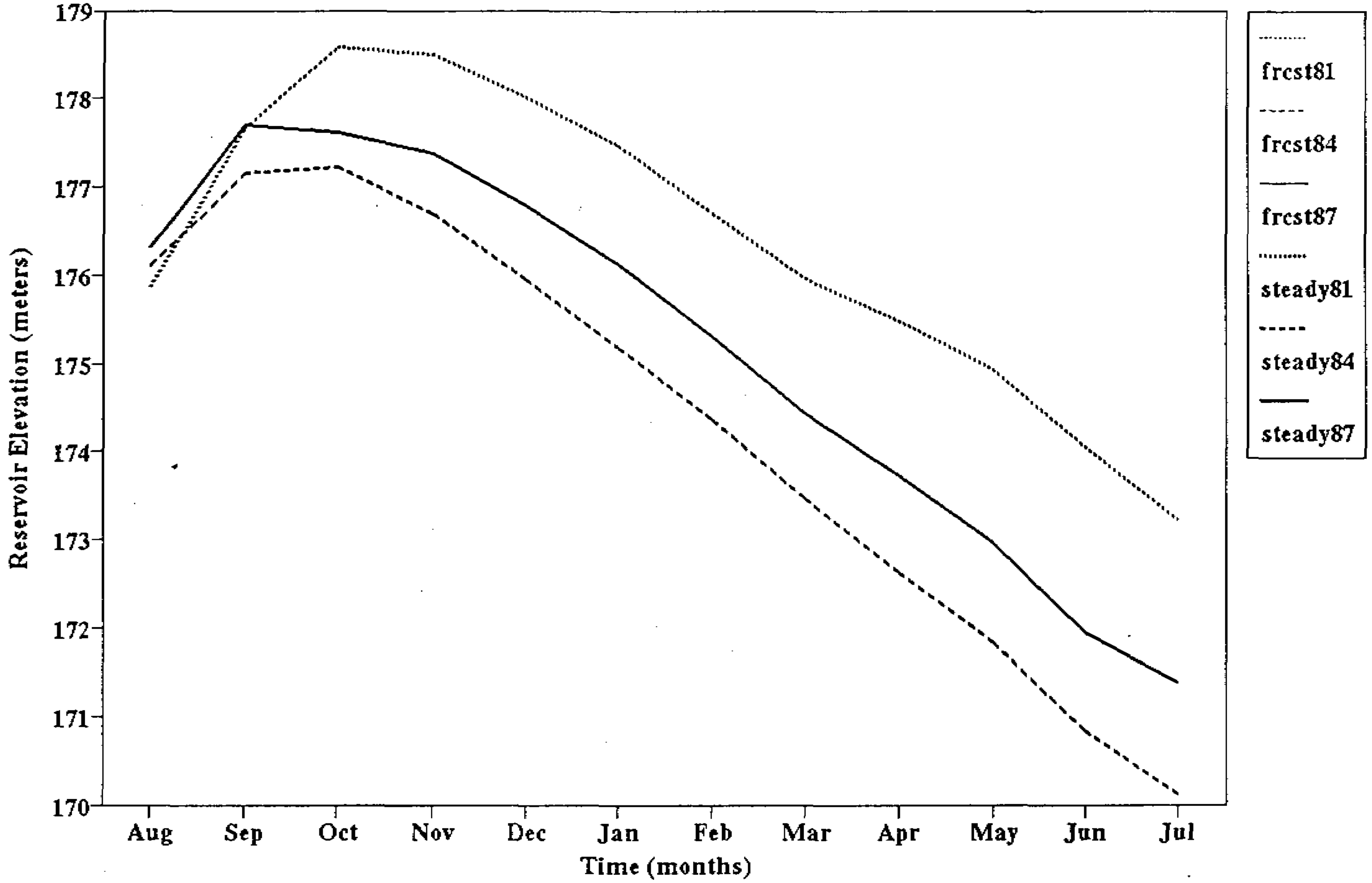

Fig. 8 Simulated Reservoir Elevations for the Forecasted Inflow Series Using Initial Elevation of 175m

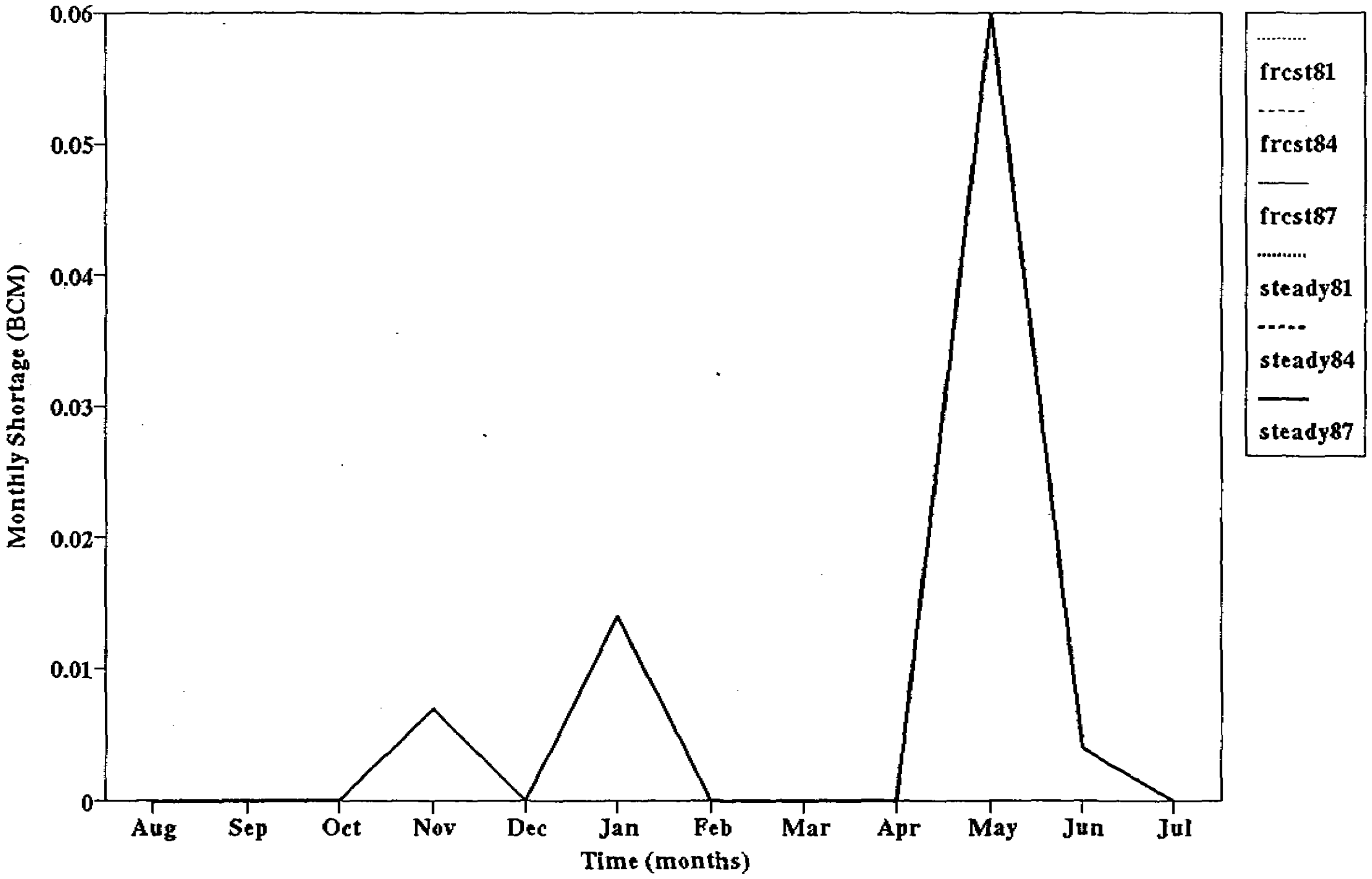

Fig. 9 Simulated Monthly Shortages for the Forecasted Inflow Series Using Initial Elevation of 175m

5. CONCLUSION

The most important variables considered to check the improvements in the policies derived by the stochastic DP were the monthly shortages that would be experienced by the reservoir system when adopting certain release policies and the reservoir storage volume at the end of each time period. It is concluded that policies obtained from the stochastic DP depend on the type of recursive equation used, discretizations of reservoir inflows, and inclusion of inflow forecast information into the formulation. Neglecting the correlation between inflows of successive periods may not yield a much worse operating policy than policy obtained considering lag-one correlated inflows because of long series of inflow records which provided accurate inflow statistics for both cases. Theoretically, including updated inflow forecasts into the stochastic DP formulation should yield improved operating policies than policies derived based on historical inflow records. Including inflow forecast information in this stochastic DP did not show significant improvements in the optimal policies produced over the steady state policies because of the large size of the reservoir under consideration.

FLOOD-FLOW MANAGENENT SYSTEM MODEL OF RIVER BASIN

Soontak Lee[1] and Sungsup Ahn[2]

ABSTRACT

A flood-flow management system model of river basins has been developed in this study. The system model consists of the observation and telemeteringsystem, the rainfall forecasting and data-bank system, the flood runoff simulation system, the dam operation simulation system, the flood forecasting simulation system and the flood warning system. The Multivariate model(MV) and Meteorological-factor regression model(FR) for rainfall forecasting and the Streamflow synthesis and reservoir regulation (SSARR) model for flood runoff simulation have been adopted for the development of a new system model for flood-flow management. These models are calibrated to determine the optimal parameters on the basis of observed rainfall, streamflow and other hydrological data during the past flood periods. The flood-flow management system model with SSARR model (FFMM-SR. FFMM-SR(FR) and FFMM-SR(MV)), in which the integrated operation of dams and rainfall forecasting in the basin are considered, is then suggested and applied for flood-flow management and forecasting. The results of the simulations done at the base stations are analysed and were found to be more accurate and effective in the FFMM-SR and FFMM-SR(MV).

ABSTRACT

Cette étude a pour objet de développer le modèle du système pour la gestion de l'eau en cas de l'inondation dans le basin fluvial.

Ce modèle du système est constitué de sous-systèmes; système d'observation et d'envoi de télégramme, système de prévision de la pluie et des datas du barrage, système de simulation de l'écoulement de l'inondation, système de l'alarme de l'inondation.

Le modèle de la régression des facteurs météolo-rogiques de la régression et le modèle de la variation de la pluie ont été utilisés comme modèle de prévision de la pluie Et puis, nous avons adopté SSARR pour la simulation de l'écoulement de l'inondation; En plus, le modèle du paramètre a été corrigé et complété grâce aux datas de la régulation du réservoir plus efficace durant l'année dernière.

Le système d'administration de l'écoulement de l'inondation avec le modèle de SSARR(FFMM-SR, FFMM-SR(FR) et FFMM-SR(MV)), dans lequel l'opération intégrale des barrages et de la prévision de la pluie dans le basin est prise en considération, est donc suggéré et adapté pour l'administration et la prévision de l'écoulement de l'inondation.

Les resultats de ces simulations fondés sur les stations élémentaires, ont été analysés nous ont paru plus exacts et plus effectifs dans le FFMM-SR et FFMM-SR(MV).

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1. INTRODUCTION

The flood-flow management and control in the river basin is mainly carried out to reduce flood damages by accurately forecasting upstream and downstream flood runoff caused by storms in the area. The overall flood management in the basin during the flood period should be performed through accurate rainfall and flood forecasting as well as integrated operation of dams in the basin. In this respect, the flood forecasting system has been established to cope with severe flood conditions in the Nakdong river basin, Korea(Lee, et al, 1987). However, the existing system model in which the integrated operation of dams in the basin is not considered is not a very accurate forecasting tool(Lee, 1988-1990 and 1989).

Consequently this study attempts to develop a new flood-flow management system model with the application of newly developed techniques of integrated optimal operation of dams and rainfall forecasting in the basin(Lee, 1992).

2. THEORETICAL STRUCTURE OF SYSTEM MODEL

2.1 System Structure

The flood-flow management system is made up of 6 subsystems to perform the flood-flow forecasting and management in the river basin. Accordingly, in this study, the flood-flow management system consists, as in Fig. 1, of an observation and telemetering system of data, a rainfall forecasting and data-bank system, a flood runoff simulation system, a dam operation simulation system, a flood forecasting simulation system and a flood warning system.

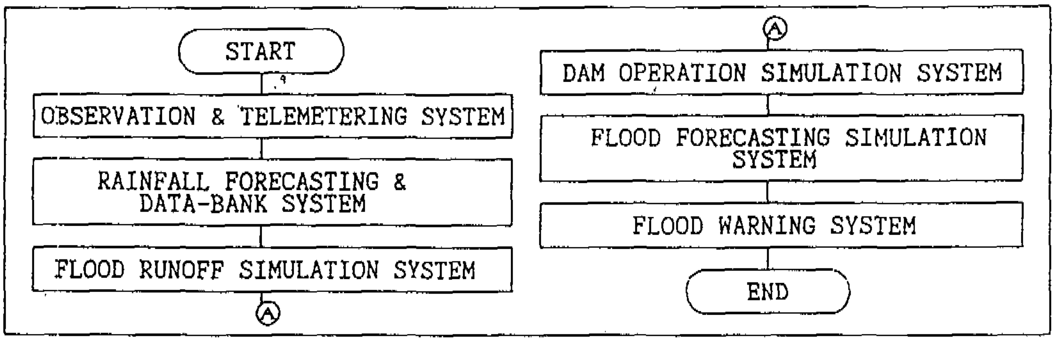

Figure 1. Structure of flood-flow management system.

In each subsystem, calibration of parameters or system simulation is carried out for the transmitted hydrologic data from T/M gauging stations, and the flood-flow forecasting and management are performed for the historical or predicted rainfall patterns according to these subsystems(Lee, 1989 and 1992).

2.2 Rainfall Forecasting Model

Two mathematical models are selected and used to predict rainfall in the river basin namely : the Meteorological-factor regression model(FR Model) and the Multivariate rainfall forecasting model(MV Model).

2.2.1 FR model

The FR model(Lee, 1992) is a regression model of meteorological factors which are believed to be the most effective components in rainfall phenomena under the assuption of their accurate prediction. The rainfall can be predicted by using the following multiple liner regression equation in the relation between meteorological characteristic factor $Z_j(j = 1, 2, 3, ..., k)$ and rainfall Y :

$$Y = \beta_0 + \beta_1 Z_1 + \beta_2 Z_2 + \beta_3 Z_3 + ... + \beta_k Z_k + e \tag{1}$$

where, e is an error in the estimation of dependent variable Y, β_0 is a constant and $\beta_j(j=1, 2, ..., k)$ is the coefficient of variable Z_j. These parameters are determined from the occurrence characteristics of storms by using the past meteorological data series, and AR(1), AR(2), ARMA(1,1) and ARMA(2,1) models are used on the basis of Box-Jenkins time series as the prediction model of meteorological factors

2.2.2 MV model

The MV model(Lee, 1992 and Johnson and Bras, 1978) is a nonstationary multivariate model for short-term rainfall prediction at multiple locations and at multiple values of time lead. In the model, vector of rainfall rates at time step t at N locations(x_N, y_N), $\hat{\imath}(t)$, is described as follows :

$$\hat{\imath}(t) = m(t) + r(t) \tag{2}$$

where, m(t) is vector of mean values at time step t, and r(t) is vector of residuals at time step t and can be defined as the following form of a diagonal standard deviation matrix $\Sigma(t)$ and a zero mean, unit variance, random vector process ε (t).

$$r(t) = \Sigma(t).\varepsilon\ (t) \tag{3}$$

At issue in this model is the dynamics of the residual term r(t). The residual is assumed to evolve in time according to a nonstationary Markov model of the form.

$$r(t+\tau) = A(t,\tau).r(t) + B(t,\tau).W(t,\tau) \tag{4}$$

where, $A(t,\tau)$ is N x N state transition matrix at time step t for a transition τ steps into the future, $W(t,\tau)$ is N x 1 vector of disturbances with zero mean value, and $B(t,\tau)$ is N x N matrix giving the effect of the noise terms at time step t on the residuals at time step $t+\tau$.

For the prediction points a measurement equation is written,

$$z(t) = q(t) - m(t) = r(t) + v(t) \tag{5}$$

where, q(t) is N x 1 vector of observed rainfall, m(t) is N x 1 vector of mean values, r(t) is N x 1 vector of true values of residual, v(t) is N x 1 measurement errors, z(t) is N x 1 vector of measured residuals and N is number of points predicted. Above equations (4) and (5) form the classic framework for the discrete Kalman filter and the filter equations can be written for a one-step lead.

The rainfall prediction at any future time $t+\tau$ is given by

$$i(t+\tau/t) = m(t+\tau) + r(t+\tau/t) \tag{6}$$

In order to implement the rainfall prediction method described above it is necessary to perform the sequence of operations which are divided into three phases : estimation of necessary statistics, estimation of system dynamics parameters and predictions, where estimations of mean and variance, covarince of normalized residuals, and storm velocity are included as the most important parameters.

2.3 Flood Runoff Simulation Model

The Streamflow synthesis and reservoir regulation(SSARR) model is selected for the flood runoff simulation in the river basin from the comprehensive studies on the applicability(Lee, 1988 - 90, 1989 and 1992). This model (Rockwood, 1968 and USAED, 1972) is a mathematical model by which streamflow can be synthesized from the evaluation of rainfall data. The model is comprised of three basic components : a watershed model, a river channel model and a reservoir regulation model. The watershed model can be explained by using the simple analogy as shown in Fig. 2. Before routing is made, the runoff components namely : surface, subsurface and baseflows are determined from the established empirical relationships of the basin characteristics in the model. These parameters are (a) Soil moisture index(SMI) and runoff percent(ROP), (b) Effectiveness of evapotranspiration (KE) and rainfall intensity(RI), (C) Baseflow infiltration index(BII) and baseflow percent(BFP) and (d) Surface(S) and subsurface(SS) separation.

The basin runoff is computed from given rainfall through the relationships in (a) and (b). The runoff is then decomposed into surface, subsurface and baseflow components through the relationship in (C) and (d).

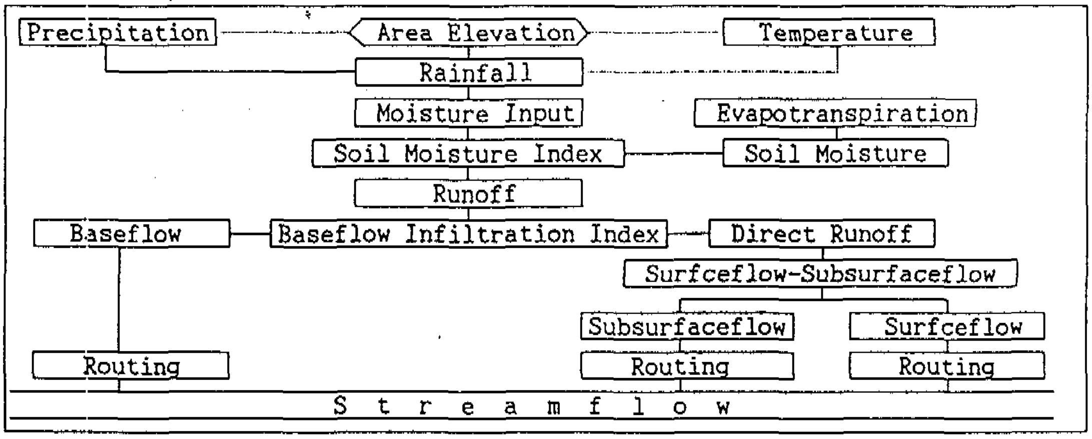

Figure 2. Conceptual diagram of SSARR model.

The three components of the runoff are then routed using the basic routing equation as follows :

$$O_2 = t.(Im - O_1)/(T_S + 0.5.t) + O_1 \qquad (7)$$

where, O_1 and O_2 are the outflows from the subreach at time t_1 and t_2, respectively, I_m is the mean inflow which is equal to $(I_1 + I_2)/2$, t is the time interval, and T_S is the time of storage. The number of subreach N and time of storage T_S are assumed for each mode of flow and they are to be determined by trial and error during calibration. The routed surface, subsurface and baseflows are then added as the outflow of a river basin.

3. APPLICATION OF FLOOD-FLOW MANAGEMENT SYSTEM MODEL AND DISCUSSIONS

3.1 Study Basin Characteristics

The Nakdong river basin, which is located in the southeastern part of Korea as shown in Fig. 3, is selected as the study basin. In this basin, there are four existing multi-purpose dams and one estuary barrage. Plus 56 T/M rainfall gauging ststions, 44 T/M water-stage gauging stations and 4 dam water-stage gauging stations are in operation for the flood-flow management and forecasting at the moment. This basin and channel reach, for the sake of operating the flood-flow management system, are subdivided in consideration of the topographic conditions of the basin, the channel conditions, the land use and the water management as shown in Fig. 3. In the division into sub-basin, the location and distribution of T/M gauging stations are also considered in tributaries and mainstream. Thus, the basin is subdivided into six sub-basins in the mainstream and eighteen sub-basins in the tributary area.

Next, the division of channel reach is performed on the reaches necessary for channel routing, which are similar in channel runoff conditions. Therefore, the channel reach of this basin is divided into nine reaches in the mainstream and six reaches in the tributaries.

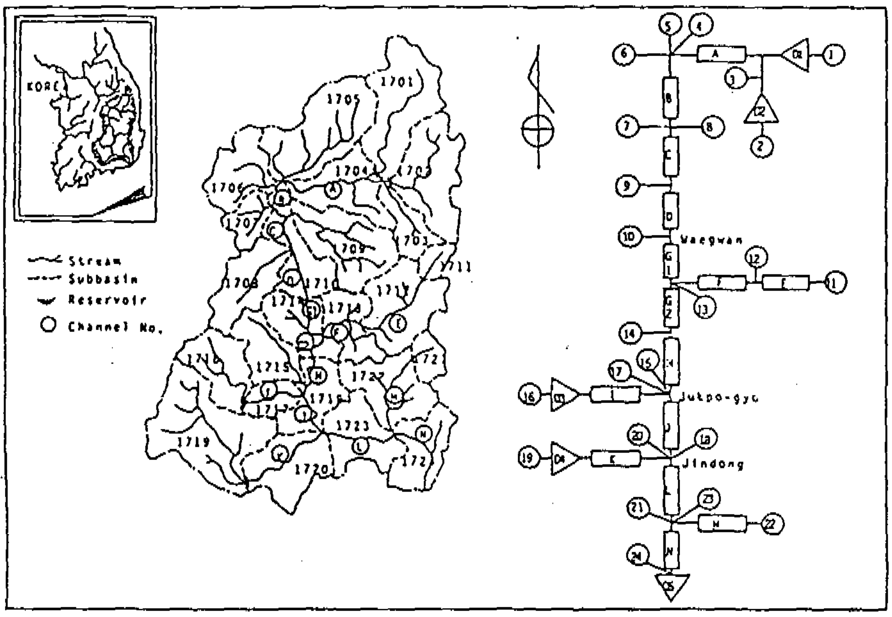

Figure 3. System diagram for flood-flow management in Nakdong riverbasin.

3.2 Model Parameters Calibration

The meteorological and hydrologic data including main storm events are selected during the wet season from June to September between 1975 and 1991, and the calibration is carried out on both model parameters of rainfall forecasting and flood runoff simulation.

3.2.1 Calibration of rainfall forecasting model

The multiple regression analysis between meteorological factors and rainfall is carried out for two types of storms, frontal-type storms and typhoon/low atmospheric pressure-type storms, to calibrate parameters in the FR model. The results of calibration show that the most effective components in rainfall phenomena among meteorological factors are 24hr-variation of atmospheric pressure, daily mean temperature, daily mean sea atmospheric pressure and cloud amount by which FR models are determined in the basin. The prediction models of these factors are also determined as ARMA(2,1) model from the calibration.

Next, the calibration on the MV model is performed to determine model parameters from the analysis of necessary statistics, storm distributions, covariance estimates, storm velocity and direction by which the MV model is determined for real-time rainfall prediction.

3.2.2 Calibration of flood runoff simulation model

In the SSARR model, it is difficult to decide the appropriate values of parameters to apply to actual situations. In particular, since the calibration of model parameters in each small basin is very difficult, three groups of sub-basins, upper, middle and lower zones, are divided to calibrate parameters such as SMI-ROP, KE-RI, BII-BFP and RGS-RS. By assuming the model parameters such as the empirical rainfall runoff relationships, the time of storage TS and the number of routing increment N in the beginning of calibration, these parameters are fed as input to the model and the simulated hourly hydrographs are compared with the observed hydrographs at various base stations. The values of these parameters are adjusted by trial and error until an adequate agreement is obtained. Satisfactory results of model calibration are obtained as shown in Fig. 4.

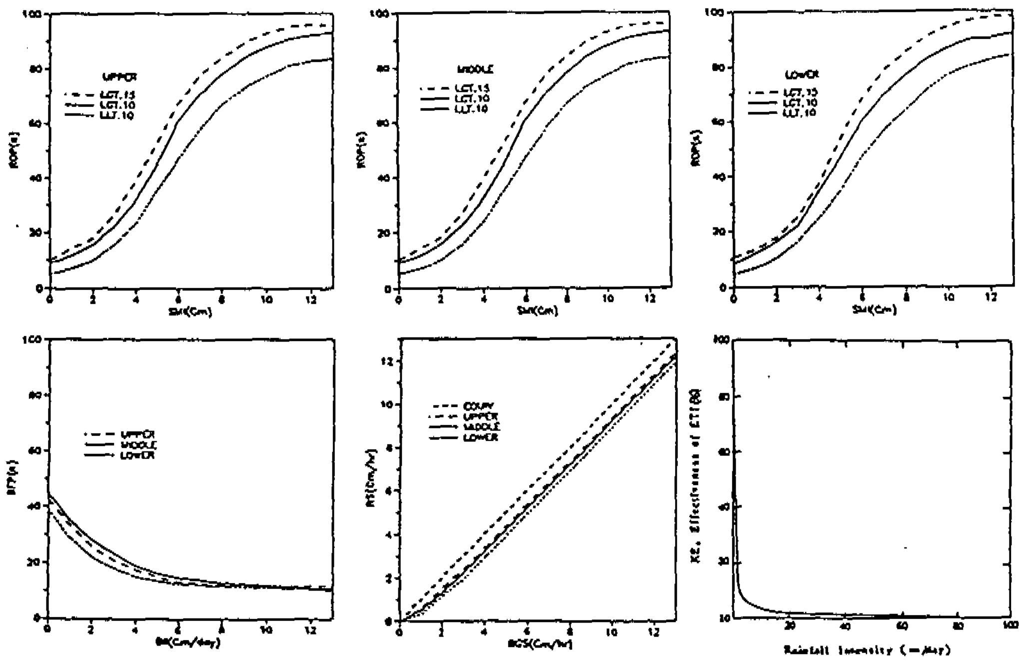

Figure 4. Optimal SSARR model parameters.

The effect of variation of these parameters is also examined by the sensitivity analysis(ME, MSE, Bias, VER and QER) from which sensitive variation of flood runoff is found according to the change of parameters as in Fig. 5.

3.3 Model Application and Discussions

The flood-flow management model with SSARR model(FFMM-SR model), in which the rigid ROM technique is adopted as the optimal integrated operation rule of dams in the river basin(Lee, 1988-90 and 1992), is applied in the flood management system in the Nakdong river basin. The forecasting or management decisions are generally done on the main storms during the flood period from June to September each year.

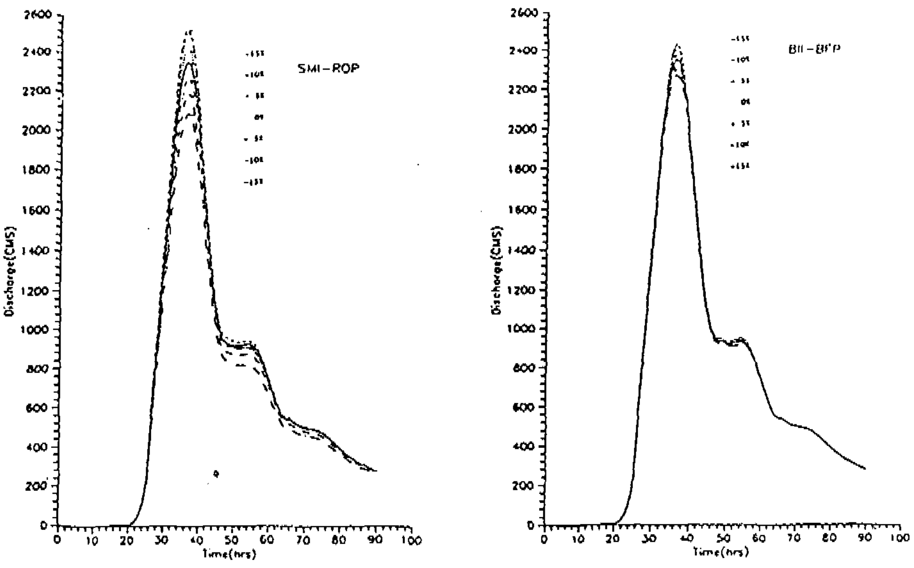

Figure 5. Effect of variation of model parameters on runoff.

The results of system management are compared with those of the existing flood-flow management models without the integrated operation of dams(FFMM model) and with the integrated operation of dams(FFMM-SF model) in both of which the Storage function model is used as the basic simulation model. The typical results of management at Jindong gauging station in the lower Nakdong river are presented as shown in Fig. 6 for the years 1990 and 1991. The management results by the FFMM-SR model show the most effective flood-flow management and integrated operation of dams in the basin.

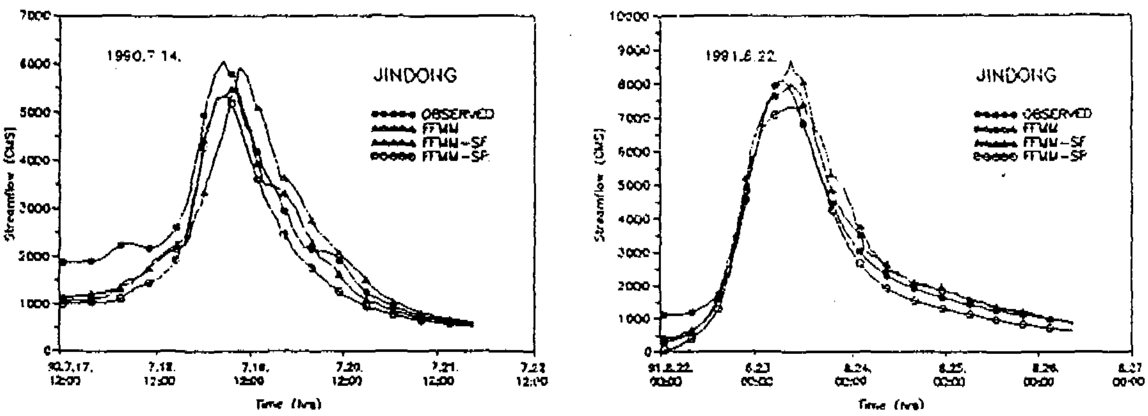

Figure 6. Flood-flow management at Jindong station in Nakdong river.

Next, this FFMM-SR model associated with the rainfall forecasting models, the FFMM-SR(FR) model in the Meteorological factor regression model(FR model) and the FFMM-SR(MV) model in the Multivariate forecasting model(MV model), are then applied to the flood-flow management system in the basin according to the process of system simulation as shown in Fig. 7. The results of management at the same gauging station are presented in Fig. 8, where the results associated with the rainfall forecasting model show a bit less of the values of flood- flow and some differences in peak time than those of FFMM-SR model.

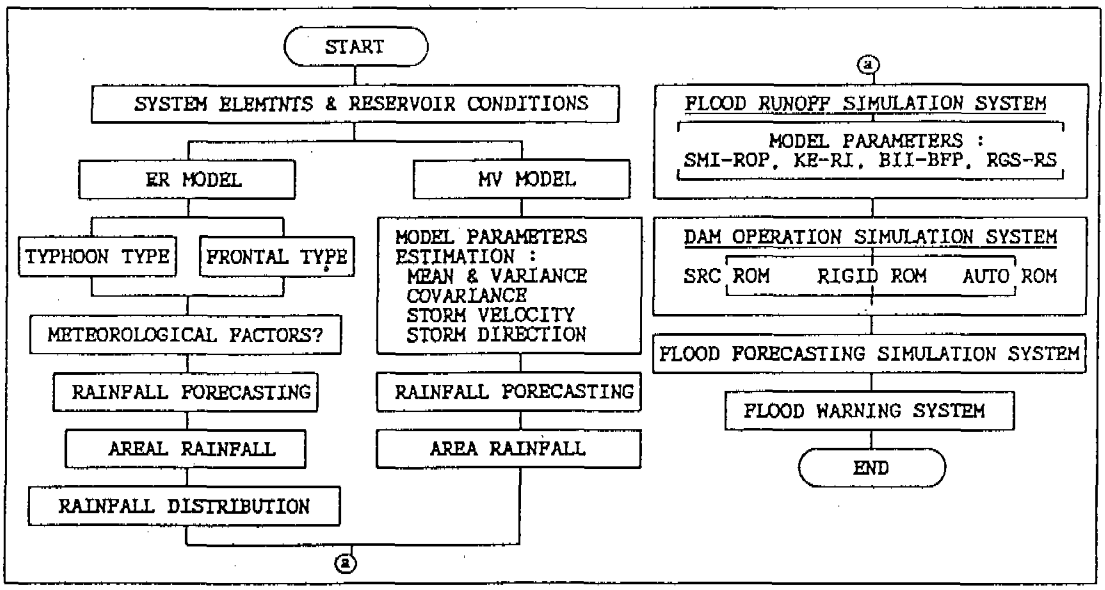

Figure 7. Simulation process of flood-flow management system associated with rainfall forecasting model.

But the results show that the flood-flow management with the rainfall forecasting show good agreement with the corresponding values of the FFMM-SR model. The FFMM-SR(MV) model resulted in more effective management than could be obtained by just following the FFMM-SR(FR) model in general.

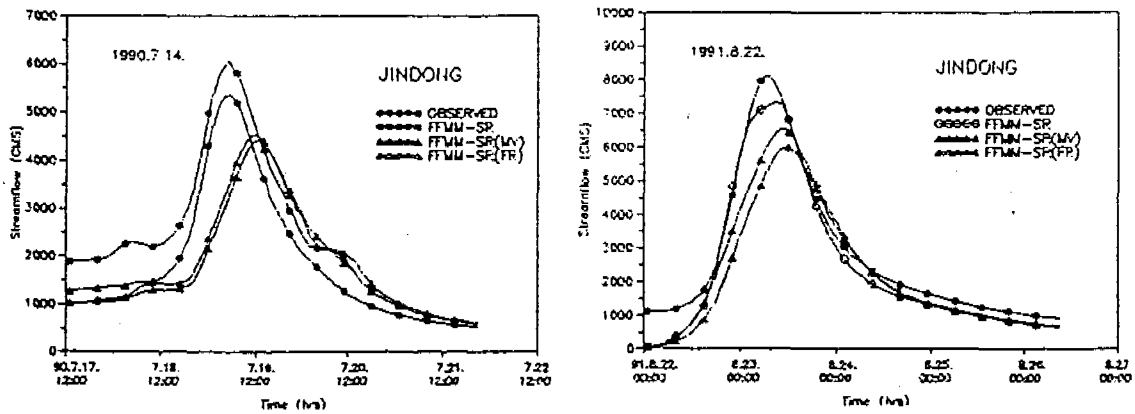

Figure 8. Flood-flow management associated with rainfall forecasting model

4. CONCLUSIONS

A flood-flow management system model and its algorithm have been developed in this study for the Nakdong river basin and the system model consists of the observation and telemetering system, the rainfall forecasting and data-bank system, the flood runoff simulation system, the dam operation simulation system, the flood forecasting simulation system and the flood warning system. The Meteorological-factor regression model(FR model) and the Multivariate model(MV model) for rainfall forecasting and the Streamflow synthesis and reservoir regulation model(SSARR model) for flood runoff simulation are chosen as the mathematical models for flood-flow management and calibrated on the basis of observed rainfall, streamflow and other hydrological data during the past flood periods between 1975 and 1991. Good results are obtained in the calibration from which optimal model parameters are determined. After the calibration, the flood-flow management model with the SSARR model(FFMM-SR model) in which the rigid ROM technique is adopted as the optimal integrated operation rule of dams is applied in the flood-flow management system and compared with the existing flood-flow management models without the integrated operation of dams(FFMM model) and with the integrated operation of dams(FFMM-SF model) in both of which the Storage function model is used as the basic simulation model. The management results by the FFMM-SR model show the most effective use flood-flow management in the basin.

Next, the FFMM-SR models associated with the rainfall forecasting models, FFMM-SR(FR) model and FFMM-SR(MV) model, are applied in the flood-flow management system. From these management results, the flood-flow management with the rainfall forecasting show good agreement with the corresponding values of the FFMM-SR model using observed rainfall values, and the FFMM-SR (MV) model provides more effective flood-flow management in general.

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REPRESENTATIVE RAINFALL PATTERN SUITABLE FOR HYDROLOGIC DESIGN IN SINAI

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ABSTRACT

Sinai is a peninsula located at the north eastern corner of Egypt. Its area is about 62,000 squared kilometers, and it is classified as arid zone. The average yearly rainfall is less than 100 mm., except between EL-Arish and Rafah where the rainfall is greater than this value. The people living in Sinai and their livestock are suffering from water shortage as the water resources are scarce and not yet well utilized. At the present time there are many attempts to study, evaluate, and utilize the water resources in Sinai. It is proposed to construct many structures to store the surface water such as dams, dikes and underground tanks; and to protect the infrastructure from flash flood hazards such as culverts, diversion channels, and embankments. Also, the groundwater in Sinai is being progressively used by digging many well fields in different regions. In order to have an accurate and economic design of structures, it is necessary to estimate properly the amount and distribution of surface run-off and infiltration water.

The shape of the rainfall hyetograph plays a significant role in defining shape and timing of the run-off hydrographs, and also the amount of run-off versus infiltration. The determination of a rainfall hyetograph is thus an integral part in the design of infrastructure used for water resources development in Sinai. Therefore, this research is oriented towards defining a rainfall pattern suitable for Sinai conditions. The available rainfall data collected by the Research Institute for Water Resources, (RIWR), Water Research Center, (WRC) were used. The data are analyzed using statistical techniques and the results are presented as hyetograph.

Storms computed using the proposed technique were compared to actual storms observed in Sinai. It was found that the observed storms and these computed have more or less the same characteristics.

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1. INTRODUCTION

Sinai peninsula is limited in its water resources. The direct rain falling over it represent the only source of fresh water. This is true because there are no rivers or canals entering Sinai, and the peninsula is almost completely surrounded by salt water bodies from all directions. Thus the detailed and complete understanding of rainfall conditions over Sinai is vital for its further development.

Many efforts have been devoted to define the total amounts of rain falling over the different parts of the peninsula. However, defining the total amounts are not sufficient. The definition of the rainfall hyetograph is equally important; the hyetograph defines the variation of storm intensity with time.

For instance, a storm with high rainfall intensity is expected to produce high runoff, which in turn will produce flash floods. Most of flash flood water in Sinai end in either the Red or the Mediterranean Sea. Thus, significant portions of the storms with high rainfall intensities will be lost, and in general can not be considered presently as useful water resources.

On the other hand, a storm with very low intensity with long duration will produce limited runoff if any. Most of the rain water will infiltrate through the soil and will ultimately recharge some groundwater aquifer. Consequently, it can be concluded that for the accurate definition of the useful water resources available in Sinai both the total amount of rain as well as the rainfall patterns are needed.

Also, the definition of the possible rainfall pattern is needed in many engineering applications. For example in designing dam capacity or the size of the spillway of a dam, the total amount of runoff as well as the peak discharges will be needed. These quantities will in turn depend on the rain fall intensity among other factors. Also, in estimating the safe withdrawal from a certain groundwater aquifer the annual recharge will be needed. Yet, the recharge will depend on the rainfall pattern. These are only few of the many applications in which the rainfall pattern will be needed.

Surprisingly enough, most of the previous efforts targeted the estimations of the total amounts of rainfall, no attempts were geared towards the evaluation of rainfall pattern for Sinai. The main goal of the present study was thus to define a reliable representative rainfall pattern that can be used by hydrologist and water resources engineers in the future. The operational objectives selected to achieve this goal are the following:

1. Define on quantitative basis a storm pattern that represents extreme events for Sinai.
2. Test the anticipated storm pattern to obtain the runoff hydrograph for selected storms.

2. DESCRIPTION OF AVAILABLE RAINFALL RECORDS

As one of its goals, the Research Institute For Water Resources (RIWR) began studying the hydrological regime through Sinai Water Resources Project which is funded by the Commission Of The European Communities (EEC). The RIWR installed weather stations (which includes rainfall measuring instruments) and rainfall stations in some particular places in such a way that there is no over lapping between the RIWR stations and the stations of the General Meteorological Authority of Egypt (GMA).

Rainfall stations were chosen such that they represent the different zones and provide areal rainfall estimates for Sinai. RIWR installed 67 rainfall gauged points all over Sinai, from which 25 rainfall gauged points in North Sinai and 42 rainfall gauged points in South Sinai. Fig. (1) illustrates a map of Sinai and the location of the rainfall gauging points all over Sinai in addition to the locations of the representative catchments.

On the other hand, the RIWR selected three representative catchments which can be considered as a sample of other watersheds that are similar in hydrological and geological characteristics. The first two representative catchments are in Wadi Sudr and Wadi Feiran in Southern Sinai, while the third is in Wadi El-Godirate in Northern Sinai. In these three representative catchments, the RIWR installed dense networks of rain gauges and runoff stations on some selected streams.

There are three types of rainfall measuring instruments used. The first one is for the total monthly rainfall measurement and is refered to as rain gauge. The other two instruments are used where the rainfall characteristics (intensity, total duration,,...etc) and the relation between rainfall and runoff are to be considered. These two instruments are those utilizing mechanical recorder and the solid state memory.

Only those storms with depth equal to or greater than the depth of the storm with 10 year return interval depth are considered in this study. Information about the 10 year return intervals were obtained from the analysis of long term records of the General Meteorological Authority of Egypt (GMA), and from the 10 year and 50 year return interval isyohetal map prepared by Helwa, (1992).

Only 39 storms from all the records of the rainfall gauging points all over Sinai were satisfying this condition. The 39 storms were from 22 rainfall gauged points and covered the period from 1987 to 1992.

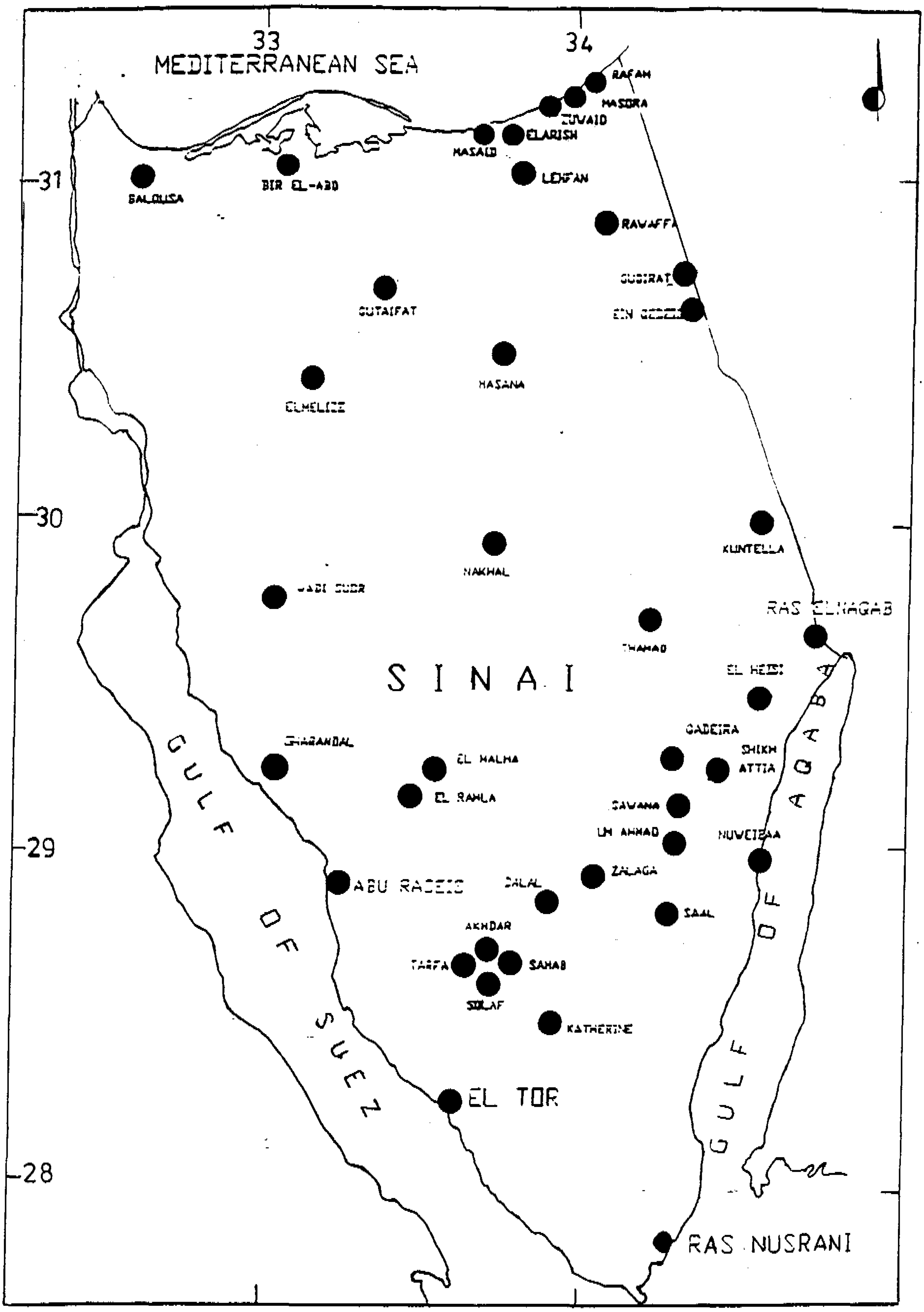

Figure (1) Location of rainfall gauged points in Sinai

(T6-S3) 4.4

3. PROPOSED RAINFALL PATTERN FOR SINAI

A design storm is a rainfall pattern defined to be used in hydrologic system design. Usually, the design storm is used as the input, and the rates of flow is the output.

Watt et.al., (1985) stated that "Defining a realistic synthetic storm pattern is not an easy task because there is a great limitation on the definition.".

Chow, (1964) mentioned that for dam design, there are three types of design storms. The first is the storage design storm which is established to determine the greatest volume of stored water that will be needed in a certain period. The second type is used to design spillway in emergency case and the third type is used to determine the free board. The duration of the second and third design storm types depends on the time of concentration of the study basin.

Since storm pattern design should be created to satisfy and represent hypothetical critical cases, a sort of storm maximization was done. Therefore the maximum 6, 12, 18, 24, 30, 60, 90 and 120 min intensity (I_{6min}, I_{12min}, I_{18min}, I_{24min}, I_{30min}, I_{60min}, I_{90min} and I_{120min}) were obtained for each storm. Dates, total depths, total durations, return periods, in addition to the maximum intensities for different durations for the storms considered are shown in Table (1).

It was noticed that a significant portion of the total depth of each storm fall in small duration related to the total duration of the storm. Thus, instead of using the total depth as an indicator form storm characteristics, Some parameters such as the duration corresponding to 70% of the total depth ($D_{70\%}$), duration corresponding to 80% of the total depth ($D_{80\%}$), etc... were used. $D_{70\%}$, $D_{80\%}$ and $D_{90\%}$
were estimated by assuming that the severe rainfall intensities will occur first an the relatively weak intensities may continue for longer durations.

The value of 70% of the total depth is divided by the duration corresponding to 70% of total depth, the intensity corresponding to 70% of total depth ($I_{70\%}$) was thus derived. Similarly, intensities corresponding to 80% and 90% of total depth ($I_{80\%}$ and $I_{90\%}$) were estimated.

Simple regression and correlation analysis was done to define which one of $I_{70\%}$, $I_{80\%}$ and $I_{90\%}$ has higher correlation with the other storm characteristics (I_{6min}, I_{12min}, I_{18min}, I_{24min}, I_{30min}, I_{60min}, I_{90min} and I_{120min}). $I_{70\%}$ was found to have better relations than the others.

TABLE (1) MAXIMUM 6, 12, 18, 24, 30, 60, 90 AND 120 MINUTES INTENSITY FOR THE ANALYZED STORMS OVER SINAI

INTENSITY AND DURATION CALCUTAION FOR SINAI

LOC	DATE	TOT. DEP. (mm)	TOT. DUR. (hr)	RET. PER.	DURATION, min 6	12	18	24	30	60	90	120
					INTENSITY, (mm/hr)							
GOD.	22/3/91	43.3	23.7	20.0	52.0	51.0	42.3	37.0	33.2	23.1	18.1	15.1
GOD.	22/3/91	34.7	22.0	15.0	50.0	38.3	31.4	26.9	24.2	17.2	14.0	12.2
GOD.	1/1/92	31.8	27.2	13.0	10.0	9.8	9.2	8.9	8.7	7.3	6.5	6.0
GOD.	12/3/90	26.5	24.2	7.0	10.6	10.1	10.0	9.0	8.4	6.9	6.3	5.8
GOD.	24/1/90	25.2	6.6	6.5	30.0	25.0	21.2	19.3	17.1	11.4	9.1	7.8
GOD.	1/1/92	25.2	27.6	6.5	11.0	9.5	8.7	8.3	7.8	6.5	5.7	5.1
RAW.	12/3/90	28.8	16.7	10.5	10.0	8.2	7.6	7.3	7.1	6.5	6.1	5.7
RAW.	25/1/90	24.9	10.0	10.0	14.0	14.0	12.3	11.0	10.2	8.0	6.9	6.1
B. ABD	21/1/89	31.6	17.9	13.0	48.0	40.8	37.6	34.2	30.6	19.3	14.8	12.1
B. ABD	21/3/91	23.6	27.4	10.0	12.0	10.5	9.8	9.5	9.2	8.3	7.3	6.5
QUT.	25/1/90	33.6	25.1	30.0	15.7	13.0	12.0	11.4	10.9	9.7	8.7	8.1
QUT.	22/1/89	26.5	24.6	10.3	11.0	10.5	9.5	8.6	8.1	6.6	5.9	5.3
QUT.	23/12/88	25.5	23.9	10.0	13.0	11.0	8.8	7.6	6.9	5.3	4.6	4.1
SUDR	21/3/91	45.4	28.5	86.0	30.0	22.5	20.0	18.5	17.6	13.9	12.4	11.2
SUDR	21/3/91	29.0	29.3	39.7	36.0	23.5	19.0	16.8	15.0	11.1	9.2	8.2
SUDR	21/3/91	28.2	29.1	37.6	40.0	29.0	24.7	21.0	18.8	13.8	10.9	9.3
SUDR	21/3/91	28.1	29.5	37.5	23.0	23.0	22.7	20.3	18.6	13.8	11.1	9.4
SUDR	5/3/91	21.9	30.2	23.7	20.0	14.5	12.3	11.0	10.2	8.1	7.0	6.2
SUDR	2/4/90	20.4	20.4	20.6	34.0	32.3	28.5	26.5	24.8	16.3	12.3	9.6
SUDR	21/3/91	18.0	29.1	17.1	21.0	17.5	14.3	12.5	11.4	8.6	7.2	6.3
SUDR	26/1/90	15.4	12.5	13.0	9.5	8.3	7.8	7.4	7.2	6.0	5.4	4.9
SUDR	26/1/90	14.5	11.4	11.7	6.0	6.0	6.0	5.9	5.7	5.2	4.8	4.5
KATH.	22/3/91	44.2	45.9	35.0	30.0	23.0	19.3	17.3	15.8	12.4	10.5	9.4
KATH.	22/3/91	42.4	37.9	32.0	26.0	20.5	17.7	15.8	14.6	12.2	10.9	9.9
KATH.	21/3/91	41.8	49.0	29.0	16.0	16.0	15.2	14.5	13.6	10.8	9.3	8.3
KATH.	21/3/91	38.6	42.9	27.0	14.0	12.8	12.3	11.8	11.4	9.5	8.0	7.3
KATH.	21/3/91	33.8	46.0	25.0	17.5	17.5	16.0	14.6	13.8	10.3	8.9	7.8
KATH.	26/12/88	20.0	6.6	14.0	21.0	19.3	17.5	16.5	15.3	11.8	9.7	8.2
KATH.	19/12/87	17.2	14.3	12.0	10.0	8.0	7.3	6.9	6.5	5.8	5.4	5.1
KATH.	26/2/92	16.5	4.2	11.0	10.0	10.0	10.0	9.8	9.3	7.8	6.9	6.2
KATH.	5/3/91	16.0	33.7	10.5	5.2	5.2	5.1	5.1	5.1	4.5	4.1	3.9
KATH.	1/1/92	14.8	3.5	10.1	15.0	14.2	13.1	12.3	11.9	9.8	8.2	6.9
KATH.	5/3/91	14.3	32.8	10.0	7.0	6.5	6.0	5.6	5.4	4.6	4.1	3.8
SAHAB	22/3/91	40.8	34.9	30.0	47.0	39.5	36.3	33.3	30.4	21.3	16.9	14.3
SAHAB	5/3/91	11.4	24.1	10.0	3.0	2.9	2.8	2.7	2.7	2.6	2.4	2.3
SAAL	22/3/91	12.0	5.0	10.0	26.5	21.5	19.5	16.8	14.8	9.8	7.4	5.8
WATIR	21/3/91	16.7	39.5	10.0	44.7	35.0	29.7	25.3	22.3	13.5	10.0	8.1
ALKANA	17/12/85	12.7	16.0	10.0	32.7	24.7	17.8	14.1	11.8	7.1	5.4	4.4
KUNTELL	21/3/91	21.9	17.5	20.0	24.7	21.8	19.2	17.8	16.6	11.9	9.5	7.9

A group of linear regression equations in the form **Log I_t=A+B Log $I_{70\%}$** where obtained; in which I_t is the intensity corresponding to the 6min, 12min, 30min, 60min, 90min, 120min. The correlation coefficients for such equations varied between 0.82 and 0.94. Fig. (2) shows the relation between $I_{70\%}$ and I_{60min} as an example.

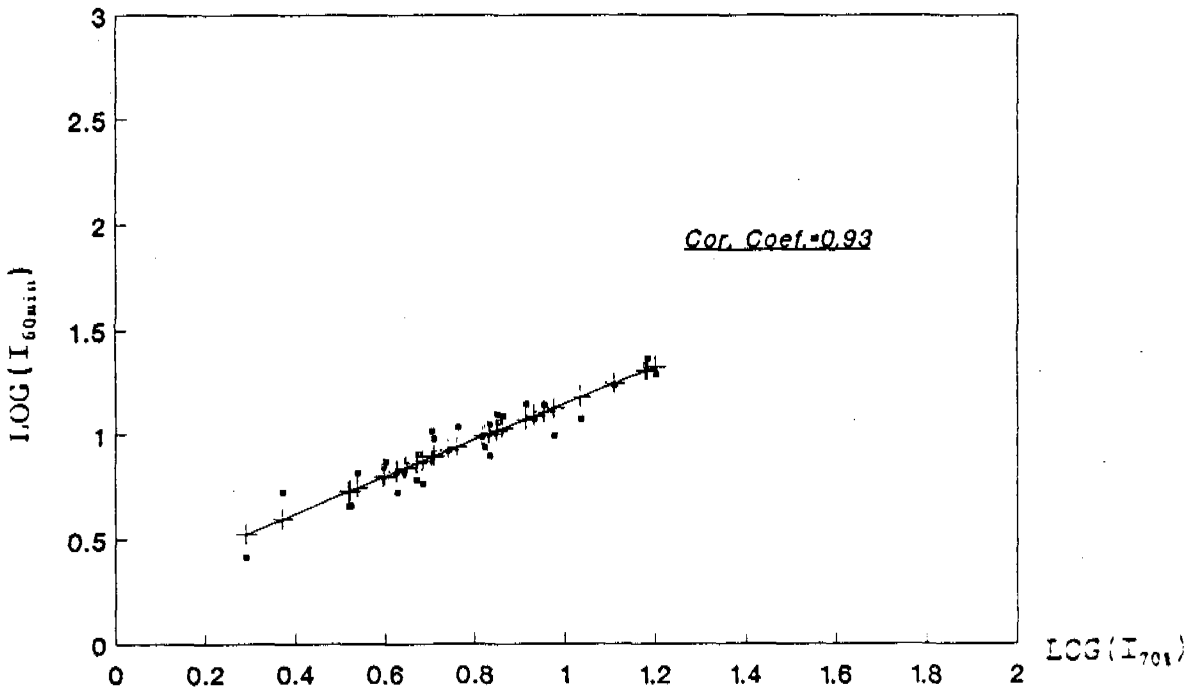

Figure (2) Typical example of the correlation between $I_{70\%}$ and the other storm characteristics

A and B are the two regression coefficients; two coefficients were obtained for each duration (6min, 12min, ... etc). These coefficients were plotted against the duration as shown in Fig. (3) and Fig. (4). A curve was fitted through the data points in each figure; and the following equations were found to represent A and B as functions of the duration t.

$$A=\exp(-0.7623-0.0078*t) \tag{1}$$

$$B=\frac{1}{(0.9961+0.01585*\sqrt{t})} \tag{2}$$

These two equations together with the relations between $I_{70\%}$ and I_{6min}, I_{12min} .... etc, are very useful. For a constant value of $I_{70\%}$, the intensity corresponding to this value on the line of I_{6min} can be estimated. The product of this I6min and duration equals 6 min (0.1 hr) will be the maximum 6 min depth. If this procedure was repeated with I_{12min} line, the maximum 12 min depth will be determined. The second maximum 6 min depth will be the result of maximum 12 min depth minus maximum 6 min depth, and so on with different duration to get the hyetograph ordinates at different durations.

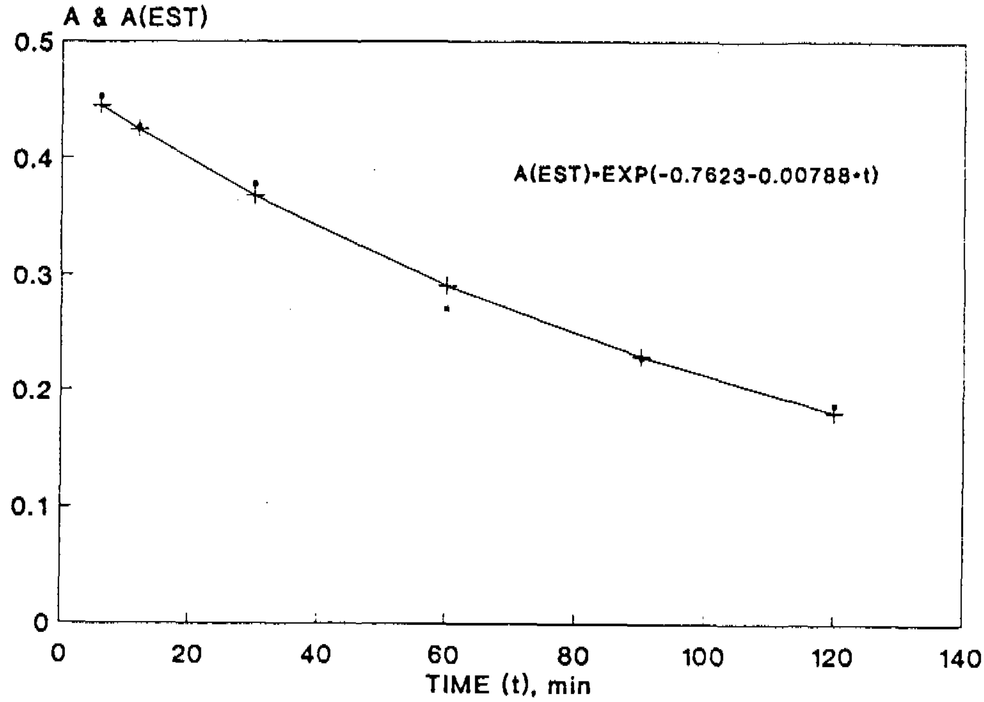

Figure (3) Relation between the regression coefficient A and the duration (t) in minutes

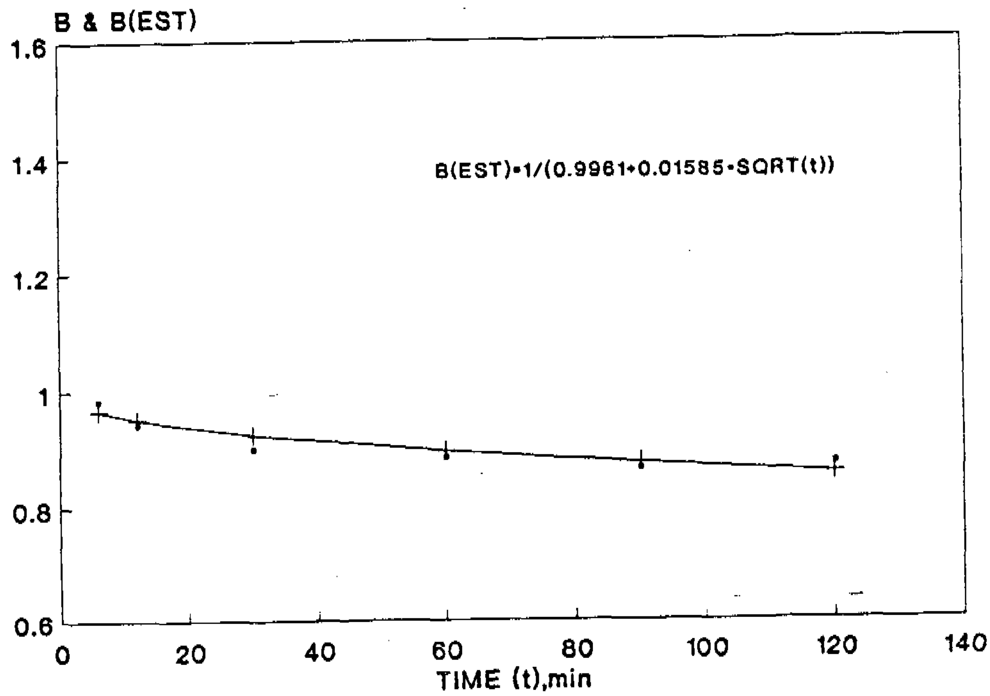

Figure (4) Relation between the regression coefficient B and the duration (t) in minutes

If this mentioned procedures were repeated with different values of $I_{70\%}$, the hyetograph ordinates can be obtained and then arranging them in descending order. Fig. (5) represents hyetograph ordinates (mm/hr) at different durations (time intervals) in min for different values of $I_{70\%}$ (10,20,30, 40 and 50 mm/hr).

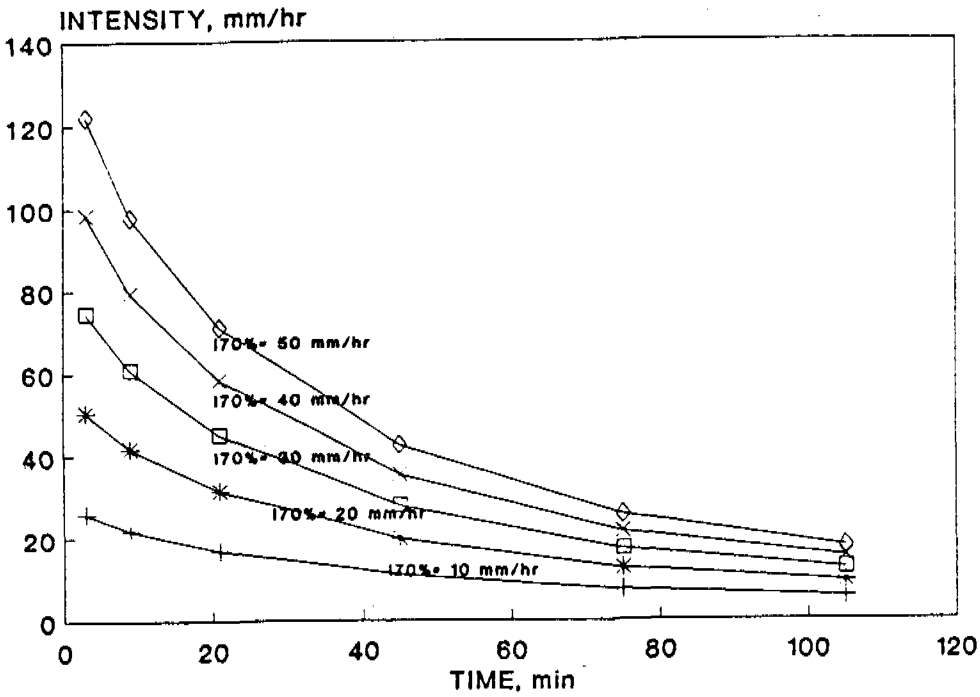

Figure (5) Estimated hyetograph ordinates of different values of $I_{70\%}$

The curves in Fig. 5 can be approximated using the following Eqn:

$$i = E * \exp(-F * \sqrt{t}) \tag{3}$$

where i represents the hyetograph ordinates in mm/hr at any time t from the beginning of the storm and arranged in descending order. E and F are constants.

Regression analysis was used to estimate the constants of this equation (E,F). The variation of E and F with $I_{70\%}$ can be expressed mathematically as in equations (4) and (5), respectively.

$$E=(-1.517613+3.879693*I_{70\%}) \tag{4}$$

and

$$F=(0.99504291+0.087662*\ln(I_{70\%}))^2 \tag{5}$$

Eqns (4) and (5) satisfy the constraints that defined by Eqn 1. The area under curves of hyetograph ordinates versus time will be equal to the total depth for any case of $I_{70\%}$ value.

$$V={}_0\int^{\infty} E*\exp(-F*\sqrt{t})\,dt=\frac{2*E}{F^2} \tag{6}$$

where V represents the total depth, mm.

then from equations (4) and (5), the total depth can be expressed as follows as a function of $I_{70\%}$ only,

$$Tot.Dep.=V=\left[\frac{(-3.035226+7.759386*I_{70\%})}{(0.99504291+0.087662*\ln(I_{70\%}))^4}\right] \tag{7}$$

Finally, equation (3) can be put in the following form with one unknown ($I_{70\%}$).

$$i=(-1.518+3.8797*I_{70\%})*\exp(-(0.995+0.088*\ln(I_{70\%}))^2*\sqrt{t}) \tag{8}$$

Equation (8) provides the hyetograph ordinates at any time. So, the intensity through any time interval can be estimated. These intensities can be arranged in the way which is considered critical for the case studied.

An attempt was done to compare the results of equation (8) with actual storms. The estimated hyetograph ordinates were arranged in the same manner as of the actual storm hyetograph ordinates. It was preferred to convert the storms from hyetograph form to the mass curve form to be easily compared. The observed and computed mass curves for two storms at Godirate and St Kathrine are provided in Fig (6) and (7), respectively.

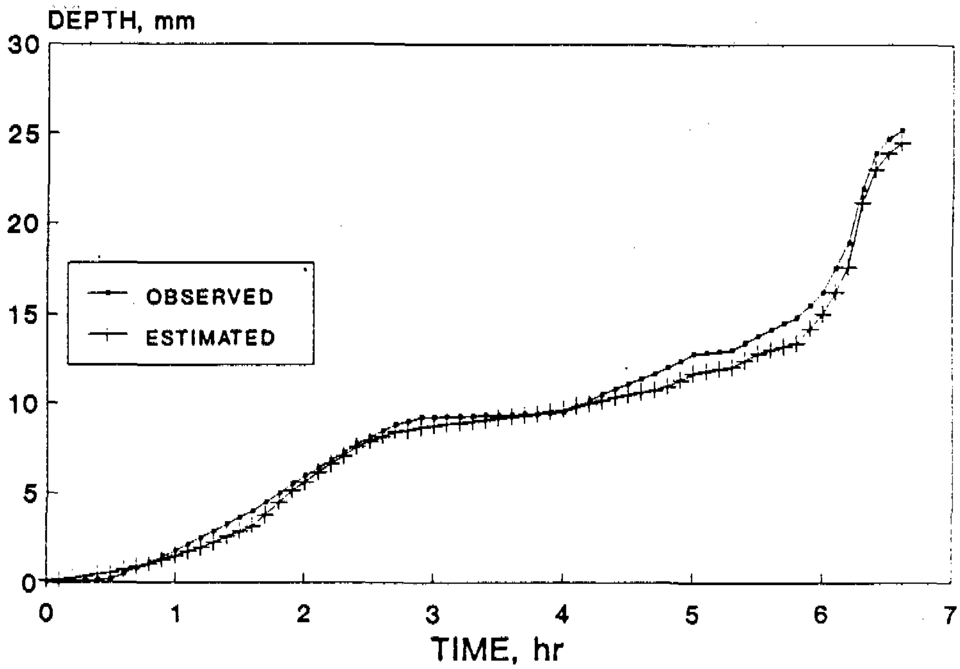

Figure (6) Observed and estimated mass curve for storm dated 24/1/90 at Godirate

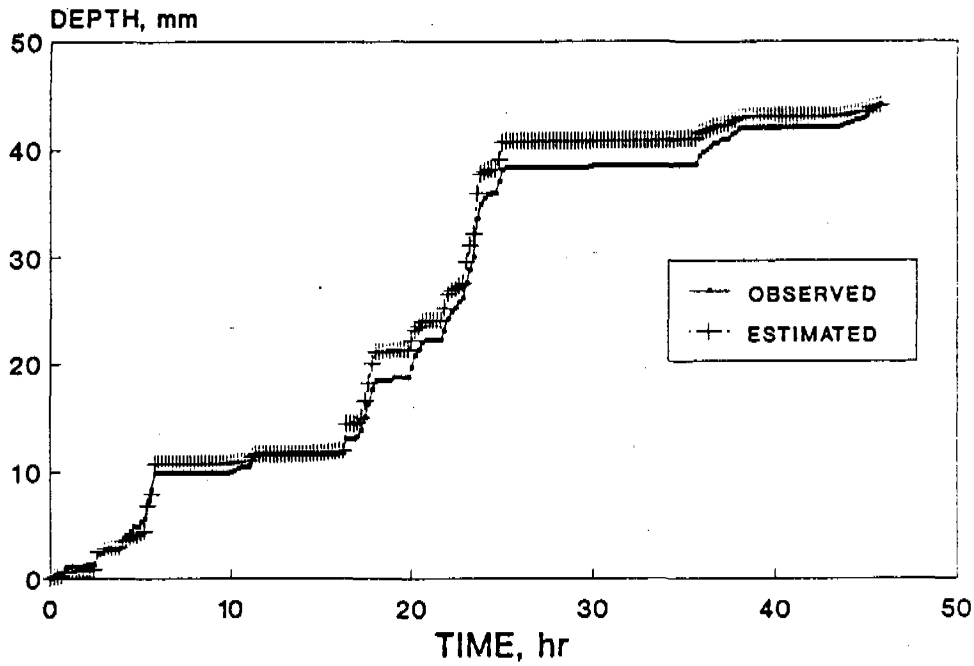

Figure (7) Observed and estimated mass curve for storm dated 21/3/91 at St. Kathrine

In summary, the following steps can be followed to estimate hyetograph ordinates to be used in runoff calculation:

1. Determining the proposed return period and the corresponding depth from the extreme analysis.
2. Get the corresponding value of $I_{70\%}$ of the desired depth.
3. Equation (8) can be used to determine hyetograph intensity ordinates at any time.
4. Calculating the average intensities for the different time intervals.
5. Arranging these intensities in the way representing the hydrological critical case to be studied.

4. SUMMARY AND CONCLUSIONS

Using statistical techniques, an equation was developed to define the design rainfall intensities for Sinai. The technique is based on actual storms observations compiled using many rainfall recording stations covering Sinai. For any duration t in hours, and return interval T in years, the equation can be used to compute the anticipated rainfall intensity with confidence. The technique developed can not be used to define the shape of the storm hyetograph, i.e, whether the hyetograph will have an early peak or late peak. However, it is worthy to indicate that both types of storms are alternatively observed in Sinai. The actual arrangement of the intensities should be assumed by the engineer according to the possible critical conditions. The output was compared with actual observed storms and the results were acceptable.

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ON-LINE IRRIGATION DEMAND FORECASTING MODEL

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ABSTRACT

In automation of multi offtake canals, it may be required to forecast the lateral canal discharges periodically for on-line operation and for updating the target pool volumes. Also, generated data of Lateral canal discharges may be required for testing, by simulation, the canal operational performance under any proposed canal automation control algorithm. For this purpose a Daily Demand Simulation Model 'DDSM' has been developed and presented in this paper. The user friendly package is made to forecast daily lateral canal discharges based on current meteorological data as well as the cropping patterns, soil characteristics and current date.

The proposed DDSM is designed to consist of four submodels, one for the generation of rainfall; another for estimation of rainfall infiltration; the third is for the estimation of reference evapotranspiration; and the fourth one is for crop evapotranspiration and daily soil water balance computations.

Data of El-Nasr canal and its branches, in Egypt, has been used for presentation of demand simulation. Figure 1 is a typical layout of EL-Nasr canal under consideration. Unfortunately, the rainfall in Egypt is very little (150mm/year). To simulate the effects of the rainfall on randomness of demands and hence on the control actions, data of Roorkee Town (India) are imposed to El-Nasr canal as an additional hypothetical demonstration example.

1. DAILY SOIL MOISTURE BALANCE MODEL

The target of any irrigation system is to supplement the water requirements of the standing crops or to make up the deficiencies due to insufficient rainfall. Daily soil moisture

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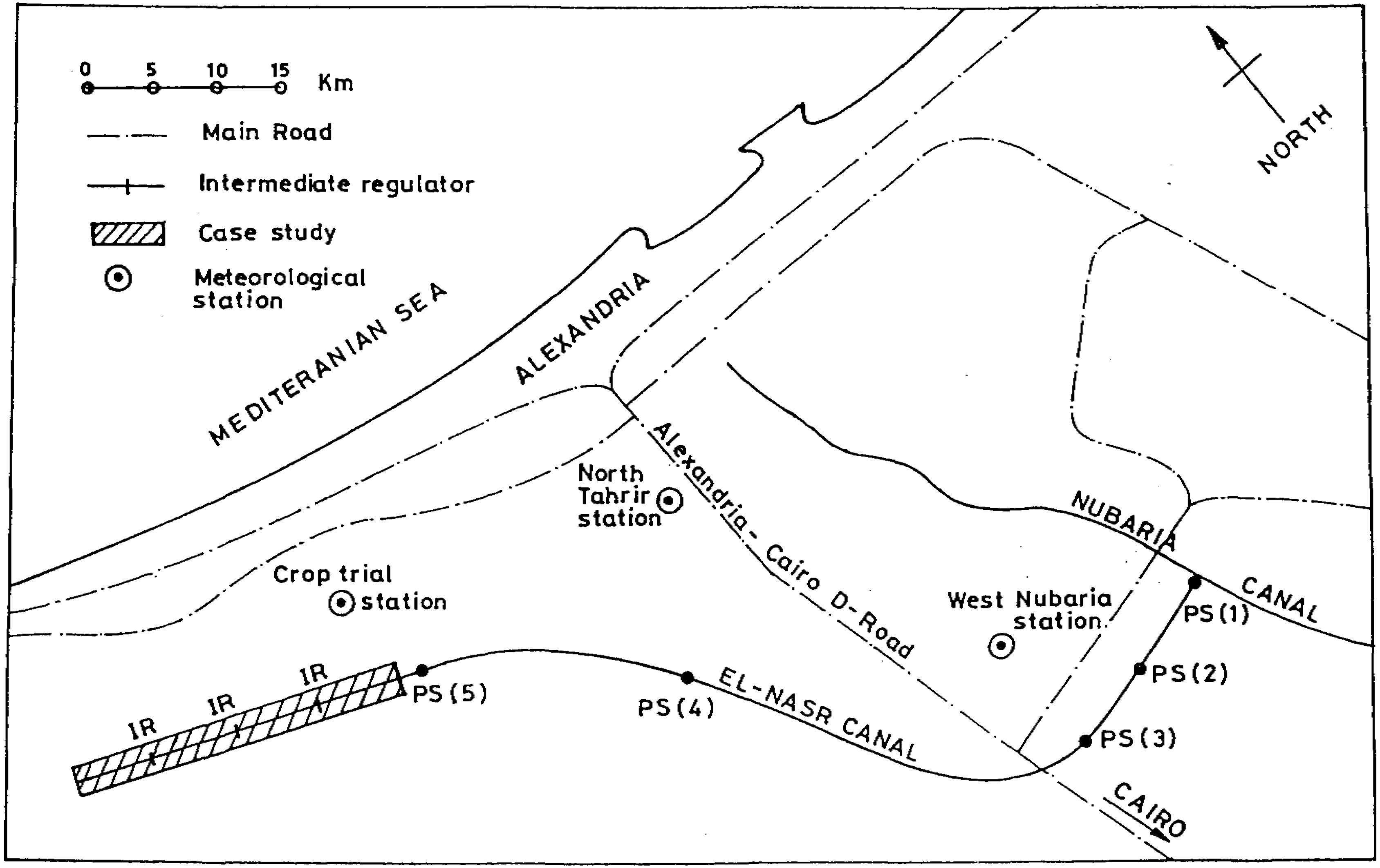


Fig. 1 Location Map of El-Nasr Canal. (Egypt).

balance model can be written in the form:

$$WD = WD + R + SWR + GWC - ETC - EVP - PER \quad (1)$$

Where WD is the equivalent soil moisture depth in the root zone at day i in mm; WD is the previous day equivalent soil moisture depth in the root zone in mm; R is the infiltrated rain from ground surface in mm; SWR is the supplementary irrigation water requirement at day i in (mm); GWC is the ground water contribution at day i in mm; ETC is the crop evapotranspiration at day i in mm; EVP is the evaporation (if any) from the ground surface at day i in mm; and PER is the deep percolation (drainage) at day i in mm.

The different terms in equation 1 require several inputs. These inputs are;

i) crop type and its sowing date;
ii) soil type and initial soil moisture at sowing date;
iii) data on the climatic variables which are required to calculate the reference evapotranspiration.

1.1 Rainfall Infiltration

Rainfall infiltrated is calculated by using the well known Mein and Larson equation presented by Slack and Larson(10). This model uses information about daily precipitation, duration of rainfall, and soil characteristics to calculate the amount of rainfall infiltration. The resulting daily rainfall infiltration is further used in the daily soil moisture balance equation to arrive at the daily supplementary irrigation water requirements of fields.

1.2 Crop Evapotranspiration

Reference evapotranspiration 'ETo' is the basic parameter for demand simulation. Several methods are available in literature (1) for computing the reference evapotranspiration 'ETo', with variable degrees of approximations and different input data requirements. In the present work, ETo is computed by using the Modified Penman's Method. This method requires input data on several climatic variables, therefore it provides the most accurate estimate of the average ETo. For demand simulation, these data are obtained from previous daily records. After computing ETo, crop evapotranspiration 'ETC ' is calculated on the basis of known crop coefficient K as follows;

$$ETC = K \; ETo \quad (2)$$

Where ETC is the crop evapotranspiration at day i in mm; ETo is the reference evapotranspiration at day i in mm; K is the crop evapotranspiration coefficient depending on the crop type and age (crop growth stage).

Crop coefficient K varies along the growth period of every crop. Similar to the procedure reported by Doorenbos and Pruit(1), the total growth period is divided into four growth stages;

i) Initial stage,
ii) Crop development stage,
iii) Mid- season stage, and
iv) Late season and harvest stage. In the absence of experimental data on the local K values the values suggested by Doorenbos and Pruit(1) have been used in the present study for El-Nasr canal and the hypothetical canal.

2. PROCEDURE

Figures 2.a and 2.b are the flow diagrams of the computational procedure followed by the algorithm 'DDSM'. Root depth is updated daily for each standing crop as a function of its current crop age. Current crop age is updated as a function of its sowing date and current date. Equivalent water depths to soil moisture at saturation, field capacity, wilting point and depletion are computed as a function of the root depth (RD) as follows;

$$WDS = C \; \theta \; RD \tag{3}$$
$$WDFC = C \; F \; RD \tag{4}$$
$$WDWP = C \; W \; RD \tag{5}$$
$$\begin{aligned} Dmin &= WDFC \; (1\text{-}P) + WDWP \; P && \text{for upland crops} \\ &= WDFC && \text{for rice} \end{aligned} \tag{6}$$

Where C is a factor accounting for unit conversion; WDS is water depth equivalent to soil moisture at saturation (in mm); WDFC is water depth equivalent to soil moisture at field capacity (in mm); WDWP is water depth equivalent to soil moisture at wilting point (in mm); Dmin is water depth at depletion limit (in mm). It is considered as a stress limit; θ is the soil porosity or total pore space (in %); F and W are the soil moisture content at the field capacity and at wilting point (in %); (in %); P is a depletion factor (less than 1.0) based on the crop and soil types as well as ETC as recommended by Doorenbos and Pruit(1).

Upper limit of soil moisture 'UP ':

This limit is used for setting a maximum soil water depth. Any excess water is considered as runoff.

$$UP = WDS + WDmax \tag{7}$$

Where UP is the water depth equivalent to upper limit of the soil moisture (in mm); and WDmax is the maximum water depth required on the ground surface (pond height), typically

RRF Total rainfall in mm/day.
RF Total infiltrated rain.
RFDU Rainfall Duration in hours.
EVA Evaporation in mm/day.
S Saturation.
FC Field capacity.
WP Wilting point.

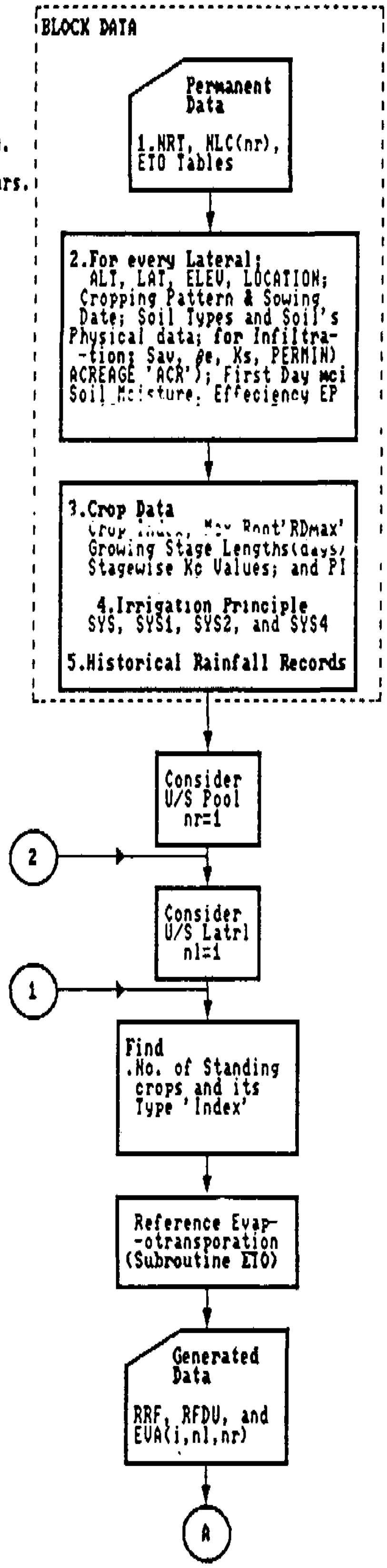
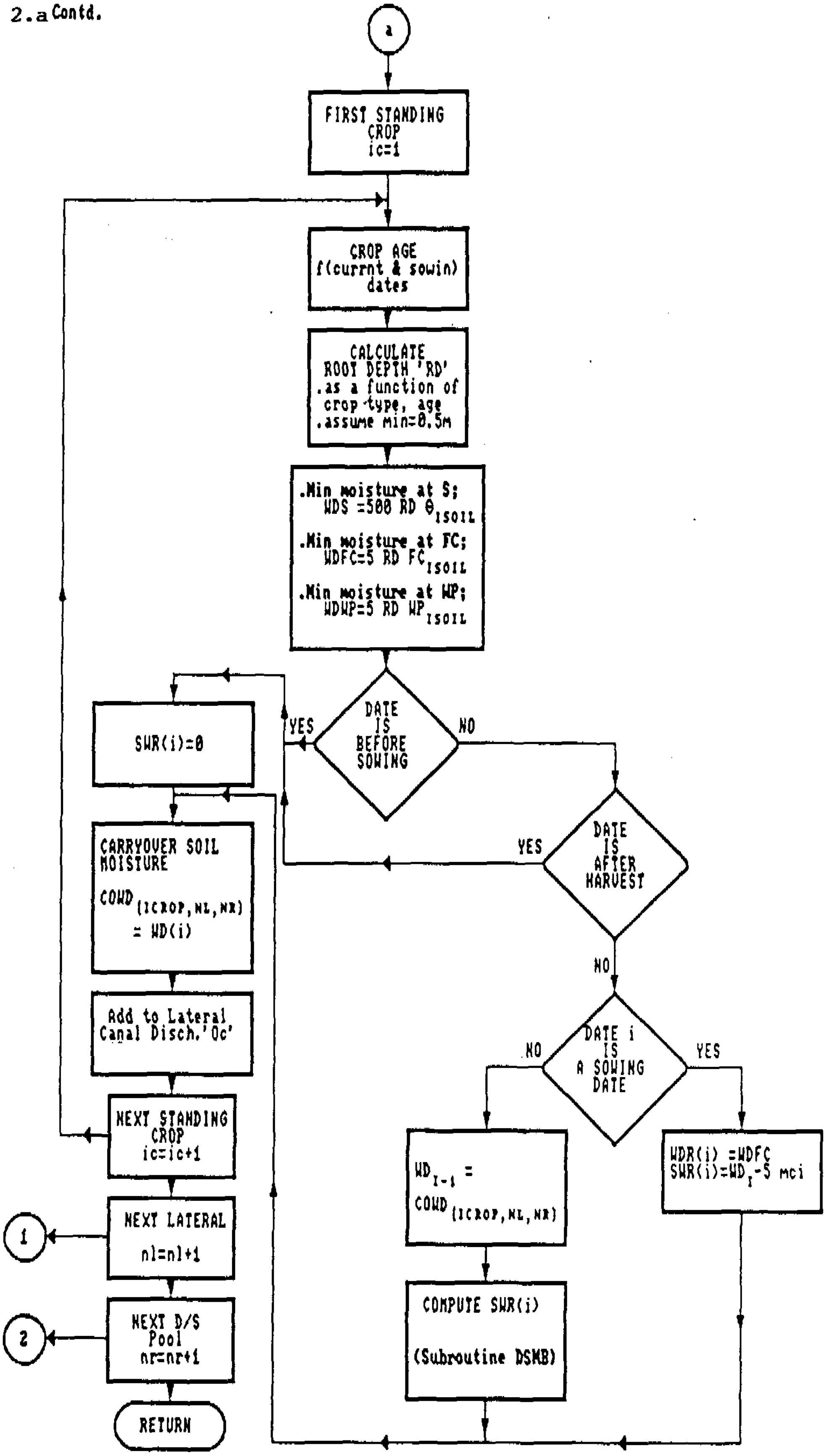


FIG. 2.a A FLOW CHART OF THE ALGORITHM 'DDSM'.

Fig. 2.a Contd.



a
FIRST STANDING CROP
ic=1
CROP AGE
f(currnt & sowin) dates
CALCULATE ROOT DEPTH 'RD'
.as a function of crop type, age
.assume min=0.5m
.Min moisture at S;
WDS =500 RD θ_{ISOIL}
.Min moisture at FC;
WDFC=5 RD FC_{ISOIL}
.Min moisture at WP;
WDWP=5 RD WP_{ISOIL}
DATE IS BEFORE SOWING
YES
NO
SWR(i)=0
DATE IS AFTER HARVEST
YES
NO
CARRYOVER SOIL MOISTURE
$COWD_{(ICROP,NL,NR)}$ = WD(i)
Add to Lateral Canal Disch.'Qc'
DATE i IS A SOWING DATE
NO
YES
NEXT STANDING CROP
ic=ic+1
WD_{I-1} = $COWD_{(ICROP,NL,NR)}$
WDR(i) =WDFC
SWR(i)=WD_I-5 mci
NEXT LATERAL
nl=nl+1
1
COMPUTE SWR(i)
(Subroutine DSMB)
NEXT D/S Pool
nr=nr+1
2
RETURN

Dmin Lower limit of soil moisture.
ETo Reference Evapotrarspiration mm/day.
ETc Crop evapotranspiration in mm/day.
ETr Equivelant required crop consumption.

GWD Groundwater depth from groundsurface.
KC Crop Coefficient.
KS Stress Coefficient.
OFL Runoff in mm/day.
PER Deep percolation in mm/day.
PERL Percolation limit in mm.
SWR Supplimentary irrigation water requirements in mm/day.
SYS
SYS1 | Variables defining the
SYS2 | target irrigation scheduling criteria)
SYS3
UP Upper limit of soil moisture.
WD Soil Moisture depth (mm).
WDS Soil moisture at saturation limit.
WDFC Soil moisture at field capacity limit.
WDWP Soil moisture at welting point limit.

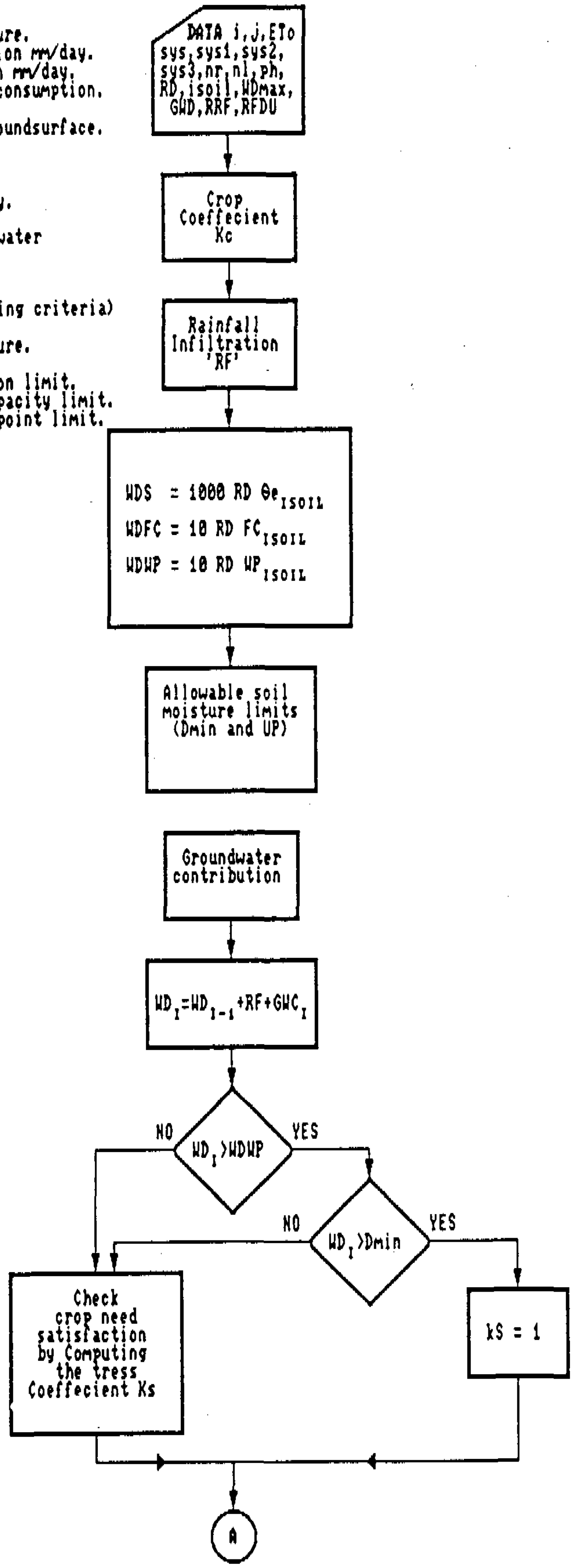


FIG. 2.b A FLOW CHART OF THE SUBROUTINE 'DSMB'.

Fig. 2.b Contd.

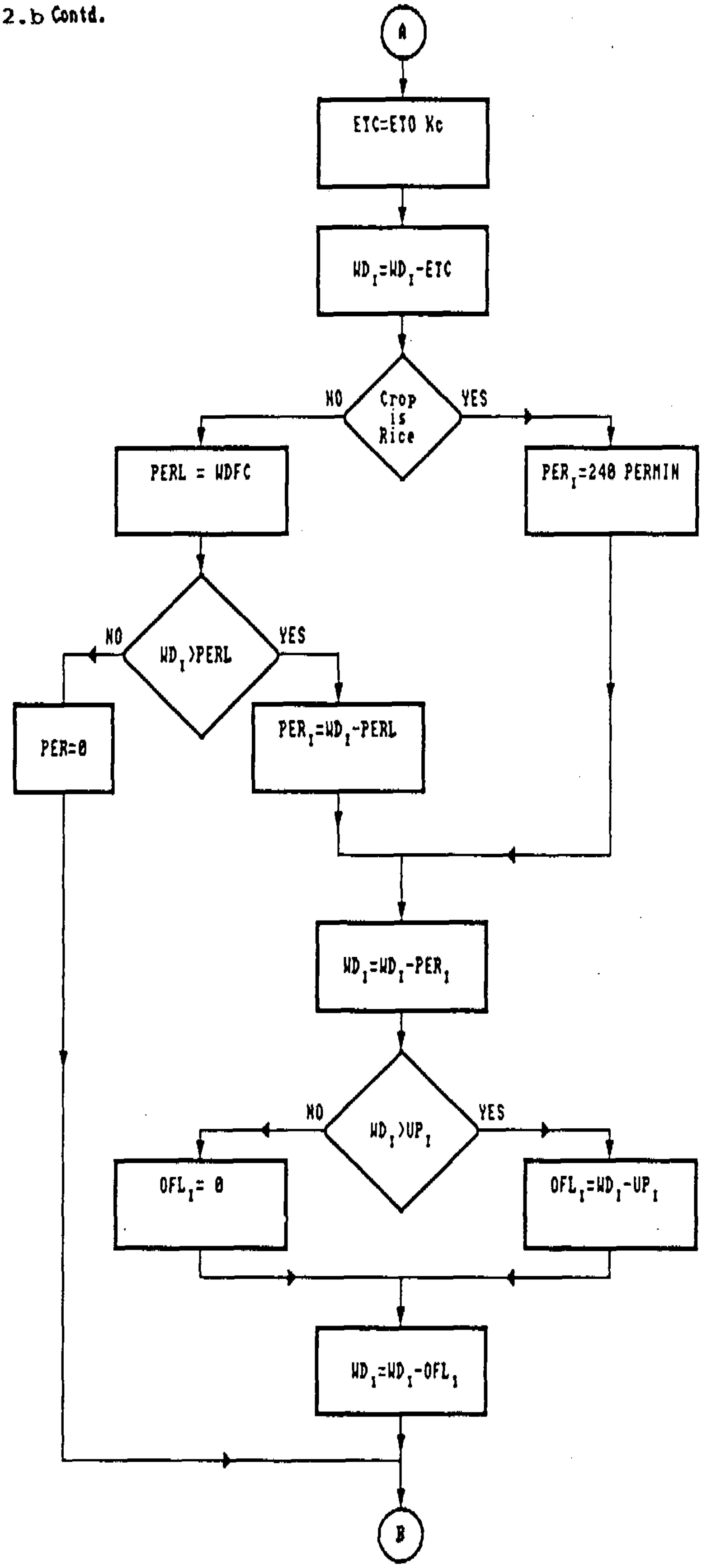
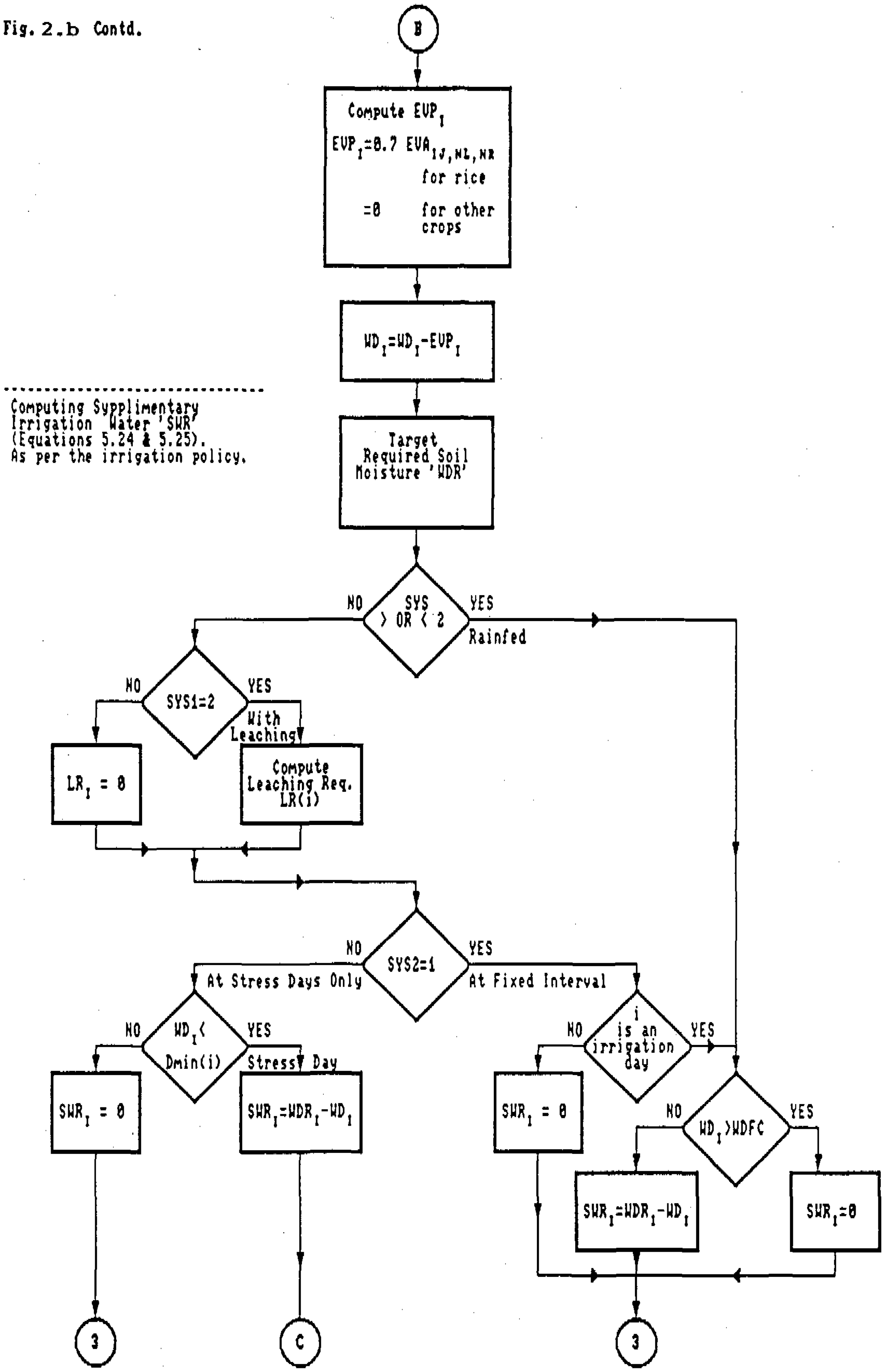

A
ETC=ETO Kc
WD$_I$=WD$_I$-ETC
NO
Crop is Rice
YES
PERL = WDFC
PER$_I$=240 PERMIN
NO
WD$_I$>PERL
YES
PER=0
PER$_I$=WD$_I$-PERL
WD$_I$=WD$_I$-PER$_I$
NO
WD$_I$>UP$_I$
YES
OFL$_I$= 0
OFL$_I$=WD$_I$-UP$_I$
WD$_I$=WD$_I$-OFL$_I$
B

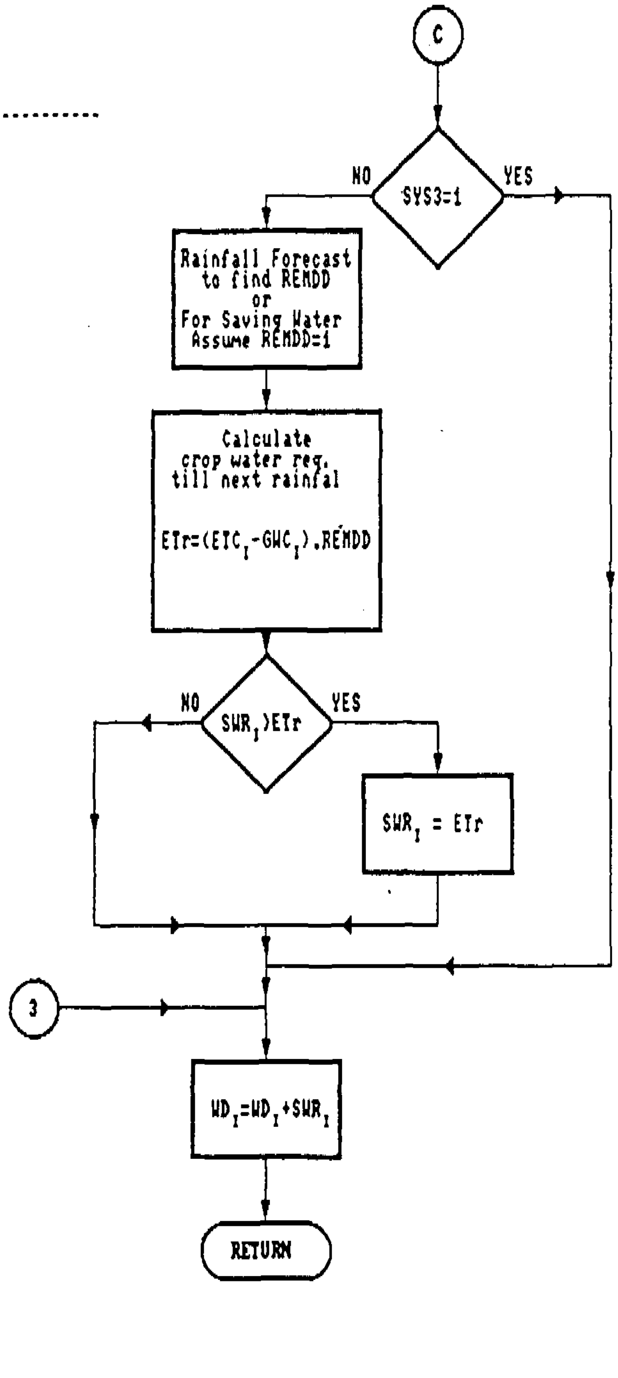
Fig. 2.b Contd.



B
Compute EVP_I
EVP_I=0.7 EVA_IJ,NL,NR for rice
=0 for other crops
WD_I=WD_I-EVP_I
Computing Supplimentary Irrigation Water 'SWR' (Equations 5.24 & 5.25). As per the irrigation policy.
Target Required Soil Moisture 'WDR'
SYS > OR < 2
NO
YES
Rainfed
SYS1=2
NO
YES
With Leaching
LR_I = 0
Compute Leaching Req. LR(i)
SYS2=1
NO
YES
At Stress Days Only
At Fixed Interval
WD_I < Dmin(i)
NO
YES
Stress Day
SWR_I = 0
SWR_I=WDR_I-WD_I
i is an irrigation day
NO
YES
SWR_I = 0
WD_I >WDFC
NO
YES
SWR_I=WDR_I-WD_I
SWR_I=0
3
C
3

Fig. 2.b Contd.

Irrigation at stress day
with amount sufficient
till next rainfall



taken as follows;

$$\begin{aligned} WDmax &= 0 && \text{for upland crops} \\ &= 150 \text{ mm} && \text{for rice (or as specified by the user)} \end{aligned} \tag{8}$$

Lower limit of soil moisture 'LL ':

This limit is the minimum soil moisture required for plant growth without any stress. It is assumed equal to the depletion limit.

$$LL = Dmin \tag{9}$$

Groundwater contribution GWC :

Ground water contribution in equation 1 is computed as a fraction of ETC , similar to the procedure adopted by Doorenbos and Pruit(1), as follows;

$$GWC = G \ ETC \tag{10}$$

Where G is a coefficient depending on the depth of ground water level below ground surface and the type of soil as given by (1).

Deep percolation PER :

Amount of water percolated below the root zone is calculated as a function of the daily soil moisture, WD . In rice cultivation, the water pond above the ground surface maintains saturated soil. Deep percolation rate in this case is taken equal to the soil permeability, subject to the condition of soil saturation. In general, a percolation limit, 'PERL ' is defined such that any soil moisture above PERL are considered as percolation. PERL can be expressed as;

$$\begin{aligned} PERL &= WDFC; && \text{for upland crops.} \\ &= WDS\ ; && \text{For rice.} \end{aligned} \tag{11}$$

Thus, deep percolation can be written for upland crops as;

$$\begin{aligned} PER &= 0 \quad ; && \text{for } WD < WDFC \\ &= WD - PERL \quad ; && \text{for } WD > PERL \end{aligned} \tag{12}$$

and the daily accumulated total deep percolation for rice (in mm/day) is calculated as;

$$PER = 240\ PERMIN \; ; \quad \text{for} \quad WD - PER \geq PERL$$
$$= WD - PERL \; ; \quad \text{for} \quad WD - PER < PERL \tag{13}$$

Where; PERMIN is the infiltration rate in rice fields, in cm/hour; PER is the deep percolation in (mm/day); and PERL is a soil moisture limit (in mm) for percolation.

Overflow (runoff) OFL :

After the tentative computation according to the water balance procedure shown in Figure 2.b, there are three possibility to account for the runoff;

1) If WD > UP overflow will take place ;

$$OFL = WD - UP \tag{14}$$

and the water depth is set equal to the upper limit.

2) If Dmin < WD < UP , the tentative water depth becomes actual depth for day i and no overflow in this case is considered.

$$OFL = 0 \tag{15}$$

3) Also, If WD < Dmin then;

$$OFL = 0 \tag{16}$$

3. DEMAND SIMULATION

DDSM computes the daily supplementary irrigation water requirements 'SWR' on the basis of the tentative water depth 'WD'. SWR is calculated to increase the soil moisture upto the target soil moisture depth. Selection of the target soil moisture 'WDR' (Water Depth Required) and selection of the irrigation scheduling criteria should be done with care. WDR has its direct effect on the amount and duration of the supplementary irrigation water requirements and then on the design capacity requirements of the canal system as well as on the irrigation interval. Thus, improper selection of WDR leads to either
i) wastage of irrigation water due to percolation;
ii) reduced rainfall efficiency by wasting rainfall into percolation; or
iii) a need for increased canal capacity.

To minimize water wastage due to deep percolation, the target soil moisture for upland crops is selected equal to the field capacity, which can be considered as the percolation limit. This is done to maintain the soil moisture sufficient to meet the crop water requirements without percolation. For rice, WDR is selected at the upper limit of soil

moisture to satisfy the pond height requirements.
WDR is computed as;

$$WDR = WDFC \quad \text{for upland crops.}$$
$$= UP \quad \text{for rice} \qquad (17)$$

The daily supplementary irrigation water requirements 'SWR ' for the farm is then computed as;

$$SWR = WDR - WD \quad \text{if } WD < WDR$$
$$= 0 \quad \text{if } WD \geq WDR \qquad (18)$$

To increase the scope of applicability of the proposed DDSM and to obtain realistic demand simulation, SWR is computed based on several alternate scheduling criterion.

The first scheduling criteria is based on the conventional rotational supply and suitable for conventionally operated canal systems. Irrigation water is applied at fixed interval (Warabandi) and the SWR is computed at the irrigation day (rotation day) only, using equation 18. The stress day coefficient 'K ' is used to compute actual crop water consumption at stress days. In these days ETC is computed by multiplying K into ETC as computed in equation 2 . K is considered a function of the root depth, soil moisture, and soil type as expressed by Jensen's(4) empirical logarithmic relationship, in the form:

$$Ks = \text{Log}\,[1+100\,(1-Dp)/Dt]/\text{Log}\,101 \quad \text{for } WD < Dmin \qquad (19)$$

$$Ks = 1.0 \quad \text{for } WD > Dmin \qquad (20)$$

Where Dp is the depleted available moisture from the root zone, expressed as;

$$Dp = Dmin - WD \qquad (21)$$

and Dt is the total available soil moisture in the root zone, expressed as;

$$Dt = Dmin - WDWP \qquad (22)$$

A second scheduling criteria could be based on maintaining the soil moisture at the target (field capacity). This method yields uniform lateral discharge, but it requires daily irrigation, except in rainy days. It does not require increased canal capacities. However, it does not make efficient utilization of rainfall. The reason is that it allows wastage of rainfall due to percolation as a result of maintaining high soil moisture by using the supplementary irrigation water. It also requires frequent irrigation. Frequent irrigation is not practical and costly if implemented without using a modern irrigation method (drip or sprinkler irrigation method).

When drip or sprinkler irrigation systems are available, maximum effective rainfall can be obtained through maintaining maximum possible pore space (air) within the soil to be always available for use by rainfall. This can be achieved by using the supplementary irrigation water to maintain a target soil moisture slightly higher (difference is in the order of ETC) than the depletion limit (stress limit). This scheduling criteria is called 'irrigation at stress days only'. A day is considered a stress day when;

$$WD \leq Dmin \tag{23}$$

Irrigation at stress days can be further based on two scheduling criterion. In the first, SWR is computed to increase the soil moisture upto the target depth 'WDR ' which can be the field capacity limit. This irrigation scheduling criteria is the conventional criteria used in practice, but it yields high water requirements and requires increased canal design capacity due to the resulting discrete and high variation in lateral discharges. In the second criteria SWR is computed at stress days, but, to meet the crop water requirements till the next expected rainy day only as:

$$SWR = (ETC - GWC) \, REMDD \tag{24}$$

Where REMDD is the expected remaining dry days (from the current day 'i' till next expected rainy day as can be obtained from the rainfall generation algorithm). During rainy season, when the probability of rainfall occurrence is high, REMDD can be assumed equal to one day for saving irrigation water.

The net irrigation water depth 'NSWR' required for any farm 'k' is calculated as;

$$NSWR = SWR \, (1 + LR) \tag{25}$$

Where LR is the percentage for leaching requirements.

Lateral canal discharge Qc is calculated by integrating the net supplementary irrigation water requirements for the various farms.

4. RAINFALL GENERATION PROCEDURE

Daily supplementary irrigation water requirements is considered a function of the daily rainfall which is random. In wet season the randomness in rainfall is more than in dry season. As a result, lateral canal demands are considered random due to the randomness in rainfall. Markov's Chain Model, Harbough and Carter(3), for rainfall process and the Monte-Carlo Generation Technique for synthetic rainfall generation are adopted for rainfall generation. The algorithm 'DRG', Daily Rainfall Generation, is made for generation of daily rainfall utilizing the historical and current rainfall records in the command areas. Procedure for rainfall generation are standard and widely discussed in literature (3). This can be summarized in the following points;

1; Input of the historical daily rainfall record of the fortnight under consideration;
2; Converting the historical rainfall data into M states (taken 20);
3; Working out the transition probability matrix (M,M) for transition from one state to another based on the occurrence frequency;
4; Working out the cumulative probability matrix

The resulting transition probability matrix provides the computational scheme for transition from any rainfall state in one particular day to the next day.

5; Synthesis of the daily rainfall state by using the Monte-Carlo Simulation technique, based on generating a random number; as follows;

   i. Compute the transition probability matrix of size (M,M).
   ii. Generate a random number 'YFL' between 0.0 and 1.0.
   iii. Initial rainfall state is calculated according to the actual rainfall state at the beginning of the fortnight (current day). It can be measured in the field once a fortnight. Alternatively, it can be generated at random by using YFL in step iii above.
   iv. Again generate a random number 'YFL'.
   v. To generate a state on NT day (where $2 < NT$). YFL is compared to the transition probability corresponding to transition from the state 'i' on (NT-1) day, which is known, to the state 'j', j+1, ...M. For each state 'j', starting with (j=1) onwards, YFL is compared with P . If YFL is less than or equal to P for a particular j then j is the output resulting state on the NT day.

6; Converting the generated rainfall state into amount of rainfall, by taking mid value of the rainfall corresponding to the generated state..
7; Repeat from step 5.iv for rainfall generation of the next day in the required fortnight period.

5. DEMAND SIMULATION - CASE STUDIES

5.1 Input Data

The following data are collected from the Ministry of Irrigation in Cairo (Egypt) for EL-Nasr canal, Halcrow(2). Soil, crop, and climatic data for the hypothetical canal are collected from the department of W.R.D.T.C. and the School of Hydrology in Roorkee University, Roorkee (U.P., India).

5.1.1 Canal system and lateral canals

In El-Nasr canal, first upstream pool contains 3 laterals, while the next pools contain

4, 4 and 2 laterals respectively as shown in Figure 3. Latitude of EL-Nasr canal is 29.75 while for Roorkee is 29.86 (considered for the hypothetical canal). Altitude (elevation) of EL-Nasr canal is 30 m above sea level and for the hypothetical canal is 268 m.

5.1.2 Cropping patterns and soil types

In EL-Nasr canal command area, two cropping seasons are followed. The first is the winter season and the second is the summer season. Typical seasonwise crops in the command area are depicted in Table 1. The same crops are used for demand simulation of all lateral canals with varying crop intensities and sowing dates as shown in Table 2. Actual cropping pattern and sowing dates of the different crops are considered in the first upstream lateral canal 'L1'. For simulation, slightly different sowing dates and crop intensities are assumed for the other laterals for simulation. These random variations are assumed to simulate actual practice in the command area where farmers may not start sowing the same crops at the same time and use the same crop intensity. Sowing dates with respect to the number of day in a year (1-365) as well as crop intensities are shown in Table 1 for EL-Nasr canal.

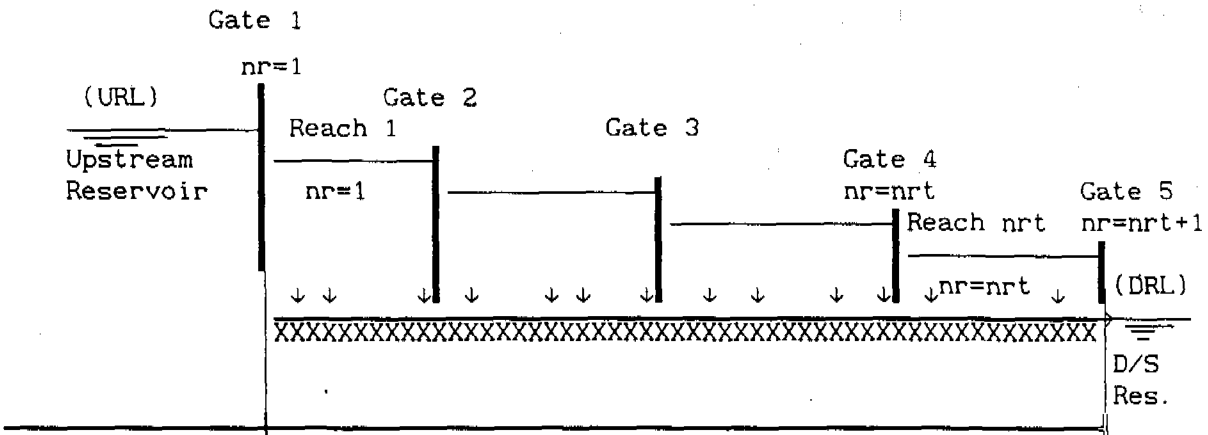
In northern India, crop pattern may be classified into (i) non-monsoon crops (ii) Monsoon crops, and (iii) Perennial crops. Typical cropping pattern is presented in Tables 3 and 4 for the hypothetical canal which considers the cropping pattern in northern India.

The soil texture in EL-Nasr canal command area is silty clay. For the hypothetical canal, the prevailing soil texture in northern India is sandy loam.

5.1.3 Meteorological data and rainfall

Actual daily data of 3 meteorological stations in the project area of El-Nasr canal are collected. The three stations are, (i) Crop Trial Station; (ii) North Tahrir station, and (iii) West Nubaria Station. Location of the three stations are shown in Figure 1. Records of daily temperature, humidity, evaporate sunshine hours, rainfall, wind speeds are collected from the three stations. Data from Crop Trail Station, Table 5, is used for the present simulation study since it is the nearest station to the project area. These daily data were assumed the same for all laterals command areas due to short length (30.5 km) of EL-Nasr canal portion under consideration. Table 6 presents the daily rainfall occurring in the project area in the year 1988 as an average operational year. From Table 6 one may conclude that rainfall is negligible.

Similarly, for of the Hypothetical canal, daily data on climatic variables and rainfall are collected for the year 1988 based on daily measurement in the school of Hydrology at the University of Roorkee. These data are summarized in Tables 7 and 8.



Chainage (Km)	55.2 62.7 70.7 77.7 85.7
Bed Level (m)	50.4 .... linear 47.35
Bed Width (m)	15 10 9 7.5
Bed Slope	.... 10 cm/km ....
Normal Average Water Depth (m)	.... 2.5 ....
Bank Level (m)	55.5 54.2 54.2 53.4
Side Slopes	.... 1 H : 2 V ....
Manning's 'n'	.... 0.016 ....
Lateral Numbers	3 4 4 2
lateral	55.7 62.67 67.17 70.64 74.24 77.37 83.0
Chainage (Km)	65.98 64.92 67.55 72.24 76.52 79.70
Bank	R R R L L R R L R L L R R
Acreage in	2370 2226 1054.2 651 4520 3217.2 2184
Hectares	3208 550.2 642.6 1008 529.2 5040
Total Area	(27300.4 Hectors)

R Right bank.
L Left Bank.

Fig. 3 Data of the Typical Longitudinal Section of El-Nasr Canal Portion Under Consideration.

Table 1 Typical Season-Wise Crops in El-Nasr Canal Command Area.

C R O P	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec
ETo mm/d	2.98	3.23	4.10	5.30	6.57	7.56	7.70	7.44	6.45	5.47	4.07	3.2
Sugarbeat (184)* (19)**												
Berseem (200) (26)												
Onion (164) (13)												
Wheat (174) (2)												
Fava Bean (153) (5)												
Potatoes (143) (16)												
Peas (106) (27)												
Cauliflo-wer(147) (28)												
Maize Gr. (122) (10)												
Maize Fud der(198) (9)												
Sunflower (102) (20)												
Watermelo (112) (12)												
Tomato (143) (22)												

* Total Period of Crop Growth Stages in Days.
** Crop Index Number Used in the Simulation Program DDSM.

Table 2   Typical Cropping Pattern in El-Nasr Canal Command Area.

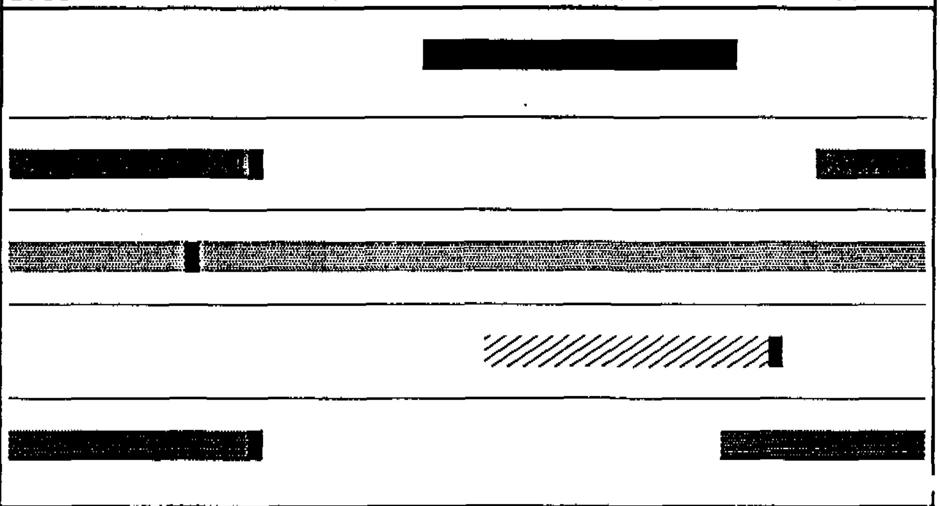
CROP i	LATERAL NUMBER L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8	L_9	L_{10}	L_{11}	L_{12}	L_{13}
2	$\frac{20}{325}$	$\frac{10}{315}$	$\frac{15}{320}$	$\frac{30}{325}$	$\frac{5}{325}$	$\frac{40}{325}$	$\frac{15}{325}$	$\frac{25}{330}$	$\frac{35}{318}$	$\frac{10}{322}$	$\frac{20}{325}$	$\frac{15}{330}$	$\frac{5}{335}$
5	$\frac{10}{290}$	$\frac{20}{280}$	$\frac{15}{298}$	$\frac{5}{290}$	$\frac{30}{290}$	$\frac{5}{290}$	$\frac{10}{290}$	$\frac{10}{285}$	$\frac{5}{292}$	$\frac{10}{292}$	$\frac{10}{290}$	$\frac{25}{285}$	$\frac{15}{277}$
9	$\frac{20}{75}$	$\frac{15}{65}$	$\frac{10}{66}$	$\frac{20}{75}$	$\frac{20}{75}$	$\frac{10}{75}$	$\frac{5}{75}$	$\frac{15}{70}$	$\frac{20}{73}$	$\frac{10}{79}$	$\frac{10}{75}$	$\frac{10}{70}$	$\frac{20}{67}$
10	$\frac{10}{153}$	$\frac{10}{140}$	$\frac{10}{139}$	$\frac{10}{153}$	$\frac{10}{153}$	$\frac{10}{153}$	$\frac{10}{153}$	$\frac{10}{150}$	$\frac{10}{148}$	$\frac{10}{148}$	$\frac{10}{153}$	$\frac{10}{150}$	$\frac{10}{151}$
12	$\frac{25}{106}$	$\frac{30}{100}$	$\frac{30}{104}$	$\frac{20}{106}$	$\frac{25}{106}$	$\frac{25}{106}$	$\frac{20}{106}$	$\frac{20}{102}$	$\frac{25}{98}$	$\frac{25}{95}$	$\frac{20}{100}$	$\frac{15}{702}$	$\frac{25}{97}$
13	$\frac{10}{260}$	$\frac{15}{250}$	$\frac{15}{255}$	$\frac{20}{260}$	$\frac{15}{260}$	$\frac{10}{260}$	$\frac{10}{260}$	$\frac{10}{252}$	$\frac{15}{245}$	$\frac{15}{257}$	$\frac{10}{260}$	$\frac{10}{252}$	$\frac{10}{258}$
16	$\frac{15}{320}$	$\frac{10}{325}$	$\frac{15}{328}$	$\frac{5}{320}$	$\frac{10}{320}$	$\frac{0}{320}$	$\frac{20}{320}$	$\frac{15}{318}$	$\frac{5}{317}$	$\frac{25}{330}$	$\frac{10}{320}$	$\frac{0}{318}$	$\frac{15}{315}$
19	$\frac{15}{275}$	$\frac{10}{260}$	$\frac{20}{270}$	$\frac{10}{275}$	$\frac{5}{275}$	$\frac{10}{275}$	$\frac{15}{275}$	$\frac{10}{278}$	$\frac{10}{269}$	$\frac{10}{267}$	$\frac{10}{275}$	$\frac{20}{278}$	$\frac{15}{265}$
20	$\frac{20}{122}$	$\frac{15}{125}$	$\frac{20}{130}$	$\frac{25}{122}$	$\frac{18}{122}$	$\frac{25}{122}$	$\frac{20}{122}$	$\frac{20}{120}$	$\frac{20}{115}$	$\frac{20}{118}$	$\frac{20}{122}$	$\frac{20}{120}$	$\frac{20}{129}$
22	$\frac{25}{123}$	$\frac{20}{120}$	$\frac{20}{124}$	$\frac{20}{123}$	$\frac{25}{123}$	$\frac{15}{123}$	$\frac{20}{123}$	$\frac{15}{126}$	$\frac{15}{117}$	$\frac{25}{114}$	$\frac{25}{123}$	$\frac{20}{126}$	$\frac{20}{116}$
26	$\frac{10}{275}$	$\frac{15}{265}$	$\frac{5}{275}$	$\frac{15}{275}$	$\frac{5}{275}$	$\frac{15}{275}$	$\frac{20}{275}$	$\frac{10}{268}$	$\frac{15}{266}$	$\frac{5}{264}$	$\frac{10}{275}$	$\frac{20}{268}$	$\frac{0}{280}$
27	$\frac{10}{290}$	$\frac{5}{292}$	$\frac{5}{285}$	$\frac{10}{290}$	$\frac{15}{290}$	$\frac{20}{290}$	$\frac{5}{290}$	$\frac{10}{295}$	$\frac{20}{284}$	$\frac{15}{285}$	$\frac{20}{293}$	$\frac{10}{295}$	$\frac{30}{287}$
28	$\frac{10}{290}$	$\frac{15}{290}$	$\frac{10}{292}$	$\frac{5}{290}$	$\frac{15}{290}$	$\frac{10}{290}$	$\frac{5}{290}$	$\frac{10}{293}$	$\frac{5}{294}$	$\frac{10}{295}$	$\frac{10}{290}$	$\frac{0}{293}$	$\frac{20}{298}$

$$\frac{\text{Numerator}}{\text{Denominator}} \text{ are } \frac{\text{Crop Intensity in \% Units}}{\text{Crop Sowing Date W.R.to Year's Days (1-365)}}$$

(day 1= 1st January)

Table 3 Typical Season-Wise Crops in Northern India (Roorkee-UP).

C R O P	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec
ETo mm/d	2.98	3.23	4.10	5.30	6.57	7.56	7.70	7.44	6.45	5.47	4.07	3.2
Rice (120)* (1)**												
Wheat (136) (2)												
Sugarcane (365) (21)												
Vegetable (120) (22)												
Berseem (183) (26)												



* Total Period of Crop Growth Stages in Days.
** Crop Index Number Used in the Simulation Program DDSM.

Table 4 Hypothetical Cropping Pattern Based on Data from Northern India (Roorkee-U.P).

CROP i	LATERAL NUMBER L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8	L_9	L_{10}	L_{11}	L_{12}	L_{13}
1	42.5/167	30/160	42.5/169	50/159	27/160	30/161	23/163	26/165	20/162	35/166	45/167	40/167	42/167
2	45.7/326	45/320	43/330	30/329	28/326	36/319	33/335	28/326	26/320	38/325	40/326	46/336	48/336
21	42.5/75	45/76	35/80	40/77	17.5/75	30/78	21/76	18/75	28/80	30/75	40/77	42/80	45/80
22	32.6/182	41/185	40/188	35/184	40/177	40/178	34/183	33/182	35/179	20/184	40/180	30/182	27/182
26	10.9/289	14/288	22/290	22/287	10/288	15/281	20/285	18/289	17/280	23/275	12/287	15/289	11/289

$\frac{\text{Numerator}}{\text{Denominator}}$ are $\frac{\text{Crop Intensity in \% Units}}{\text{Crop Sowing Date W.R.to Year's Days (1-365)}}$

(day 1= 1st January)

Table 5 Climatic Data - Crop Trial Station.

Station: Crop Trial

Latitude: 29° 44 N Longitude : 30° 53
Altitude: 30m
Period: 1979-1983

MONTH	Air Tepm. Degrees C		Relative Humidity		Evap. Class A pan	Sun-shine	Rain-fall	Wind Speed Km / day	
	Min C°	Max C°	Min %	Max %	mm /day	Hours /day	mm / month	Av. *	R **
1	5.58	17.05	63.68	88.76	2.70	6.60	31.92	180.9	1.39
2	6.92	17.44	66.41	92.10	3.23	6.50	57.00	188.0	1.87
3	9.44	21.20	62.00	92.50	4.77	8.36	31.60	218.5	2.04
4	11.13	25.46	58.42	93.80	6.52	8.23	2.23	250.0	1.92
5	13.66	27.88	51.78	91.20	7.64	10.23	—	238.0	2.30
6	16.73	30.38	53.11	93.43	9.32	12.10	—	244.0	2.30
7	19.25	30.55	56.47	92.06	9.25	12.08	—	250.4	2.80
8	19.40	31.10	59.46	92.80	9.37	11.46	—	220.9	3.02
9	17.51	29.36	60.26	92.83	8.07	10.13	—	221.8	2.65
10	13.48	28.14	56.80	94.33	5.60	9.30	8.00	165.2	2.97
11	9.11	23.29	58.97	88.84	3.95	7.17	13.06	165.0	1.80
12	8.09	19.49	54.89	85.20	3.25	6.44	15.73	154.8	1.26

Table 6 Typical Rainfall Record in El-Nasr Canal Command area in mm for the Average Year (1988).

d	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec
1	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0
6	0	0	0	0	0	0	0	0	0	0	0	0
7	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	4.0	0
11	15.0	0	0	0	0	0	0	0	0	5.0	0	0
12	0	20.0	14.0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0
14	0	0	0	0	0	0	0	0	0	0	0	0
15	0	0	0	0	0	0	0	0	0	0	0	0
16	0	0	0	0	0	0	0	0	0	0	8.0	0
17	0	25.0	0	0	0	0	0	0	0	0	0	0
18	0	0	0	0	0	0	0	0	0	0	0	0
19	0	0	0	0	0	0	0	0	0	0	0	0
20	0	0	0	0	0	0	0	0	0	0	0	0
21	0	0	0	0	0	0	0	0	0	0	0	0
22	0	0	0	2.23	0	0	0	0	0	8.06	0	0
23	0	0	17.6	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	3.73	0
25	0	0	0	0	0	0	0	0	0	0	0	0
26	0	0	0	0	0	0	0	0	0	0	0	0
27	16.95	0	0	0	0	0	0	0	0	0	0	0
28	0	12.0	0	0	0	0	0	0	0	0	0	0
29	0	0	0	0	0	0	0	0	0	0	0	0
30	0	-	0	0	0	0	0	0	0	0	0	0
31	0	-	0	-	0	-	0	0	-	0	-	0

d is the day number

Table 7 Climatic Data - Northern India.
Station: Hydrology Deptt, Univ. Of Roorkee, India.

Latitude: 30^{o} 46 N Longitude : 29^{o} 42
Altitude: 24 m
Period: 1976-1978

MONTH	Air Tepm. Degrees C		Relative Humidity		Evap. Class A pan	Sun-shine	Rain-fall	Wind Speed Km / day	
	Min C^{o}	Max C^{o}	Min %	Max %	mm /day	Hours /day	mm / month	Av. *	R **
1	8.15	21.46	40.00	95.0	1.10	6.56	0.0	16.7	.11
2	10.76	24.29	22.00	94.0	1.46	6.75	19.8	65.5	.61
3	14.04	26.72	24.00	92.0	1.57	7.64	45.9	44.6	.43
4	20.43	35.49	20.00	64.0	6.44	8.92	12.5	77.8	.69
5	25.29	40.39	15.00	55.0	6.22	9.09	6.4	111.5	.95
6	27.85	36.55	23.00	95.0	6.96	7.21	67.7	8.6	.10
7	26.64	31.97	63.00	98.0	3.13	5.45	478.1	50.2	.56
8	26.14	30.99	58.00	100.0	2.54	5.69	491.0	11.2	.13
9	25.20	32.73	52.00	98.0	3.53	6.55	281.5	20.2	.23
10	18.16	31.26	43.00	96.0	2.65	7.42	0.0	0.0	.00
11	12.25	27.15	48.00	88.0	2.24	7.78	0.0	0.0	.00
12	8.74	19.90	11.30	99.0	1.01	5.95	68.4	41.8	.47

* Represents the Average Day-time Wind Speed.
** Represents the Average ratio Between Day/Night Wind Speeds.

Table 8 Typical Rainfall Record In Northern India (Roorkee City-1988) in mm.

d	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec
1	0	0	0	0	0	0	0	74.3	0	0	0	0
2	0	0	0	0	0	3.0	0	3.0	0	0	0	0
3	0	0	0	0	0	0	42.0	23.6	0	0	0	0
4	0	0	0	0	0	0	42.2	.2	0	0	0	0
5	0	0	0	0	0	.5	135.0	15.6	0	0	0	0
6	0	0	0	0	0	0	41.0	0	0	0	0	0
7	0	0	0	0	0	0	31.0	0	0	0	0	0
8	0	0	.7	0	.7	0	1.0	38.8	12.6	0	0	0
9	0	0	6.4	0	3.0	0	0	25.4	16.0	0	0	0
10	0	0	0	0	0	0	0	37.6	0	0	0	0
11	0	0	12.0	0	0	0	0	0	0	0	0	0
12	0	2.5	0	0	0	0	1.8	1.8	0	0	0	0
13	0	1.0	0	0	0	0	6.2	11.8	0	0	0	0
14	0	5.0	10.6	0	0	0	8.6	14.4	0	0	0	0
15	0	0	0	0	0	0	1.8	40.2	0	0	0	0
16	0	0	8.0	.0	.0	1.0	1.6	0	0	0	0	0
17	0	0	0	0	0	.2	.8	0	0	0	0	0
18	0	0	0	0	2.5	7.2	.0	49.8	0	0	0	0
19	0	0	0	0	.2	5.8	2.0	7.6	0	0	0	0
20	0	0	0	12.5	0	0	16.0	4.6	.2	0	0	2.5
21	0	0	0	0	0	0	1.3	0	4.0	0	0	0
22	0	2.5	0	0	0	0	11.5	5	32.9	0	0	1.5
23	0	.5	0	0	0	36.2	.0	14.0	39.8	0	0	29.0
24	0	0	0	0	0	0	59.0	64.0	139.0	0	0	34.4
25	0	0	0	0	0	0	2.0	63.8	0	0	0	.2
26	0	0	0	0	0	0	.0	0	37.0	0	0	0
27	0	0	8.2	0	0	0	21.4	0	0	0	0	0
28	0	8.3	0	0	0	0	6.3	0	0	0	0	0
29	0	0	0	0	0	0	.0	0	0	0	0	0
30	0	-	0	0	0	13.8	1.0	0	0	0	0	.4
31	0	-	0	-	0	-	44.6	0	-	0	-	.4

d is the day number

5.1.4 Tabulated block data

This includes i) Data on maximum possible sunshine hours, Doorenbos and Pruit(1); ii) Coefficients for computing radiation terms; iii) Data for computing correction factor of ETo, Doorenbos and Pruit(1); iv) Calibrated soil data of 6 common soil classes; v) Date for computing initial crop coefficients just after sowing date, Doorenbos and Pruit(1); vi) Stagewise crop coefficients for 29 crops (1); vii) Stagewise growth period for each crop, in days, (1) ; viii)Maximum root length for the considered crops, in meters, (1) ; and ix) Depletion coefficient 'P' for computing minimum soil moisture limit.

6. SIMULATED DAILY LATERAL CANAL DISCHARGES

The computer program "DDSM" is run considering each of the three different irrigation scheduling criterion mentioned above for EL-Nasr canal and for the hypothetical canal. Resulting discharge requirements of one lateral 'L1' (first upstream lateral in the first upstream pool) is presented as a result samples in Figures 4, 5 & 6.

6.1 Irrigation Scheduling Criteria

In Figures 4, SWR is calculated only at stress days only. Supplementary irrigation water requirements are calculated, by using equation 18, to increase the soil moisture to the field capacity limit. High lateral demand is simulated as a result of the high difference between the stress (depletion) limit and the field capacity limit.

The simulated discrete demands require increased canal capacities. It also does not make best use of expected rainfall. Further, these discrete and frequent demands require discrete operation of the lateral canals, in which the canal may be required to be operated one day and closed for more than a fortnight. This leads to water wastage due to in-pool storage (seepage and evaporation loss). Such discrete operation may create several canal operation problems. Therefore, using this irrigation scheduling criteria is not recommended.

6.2 Irrigation Scheduling Criteria - 2

To economize the irrigation system by allowing the lateral canal to be operated continuously, supplementary irrigation is computed daily to maintain the soil moisture at the field capacity (using equation 18). Figures 5.a and 5.b show the reduction in the maximum lateral discharges as compared to those in Figure 4. However, it yields annual requirements which are more than the requirements based on irrigation scheduling criteria-1 presented above. Increase in the total annual water requirements in this case is due the very poor utilization of rainfall, because of maintaining high soil moisture (field capacity

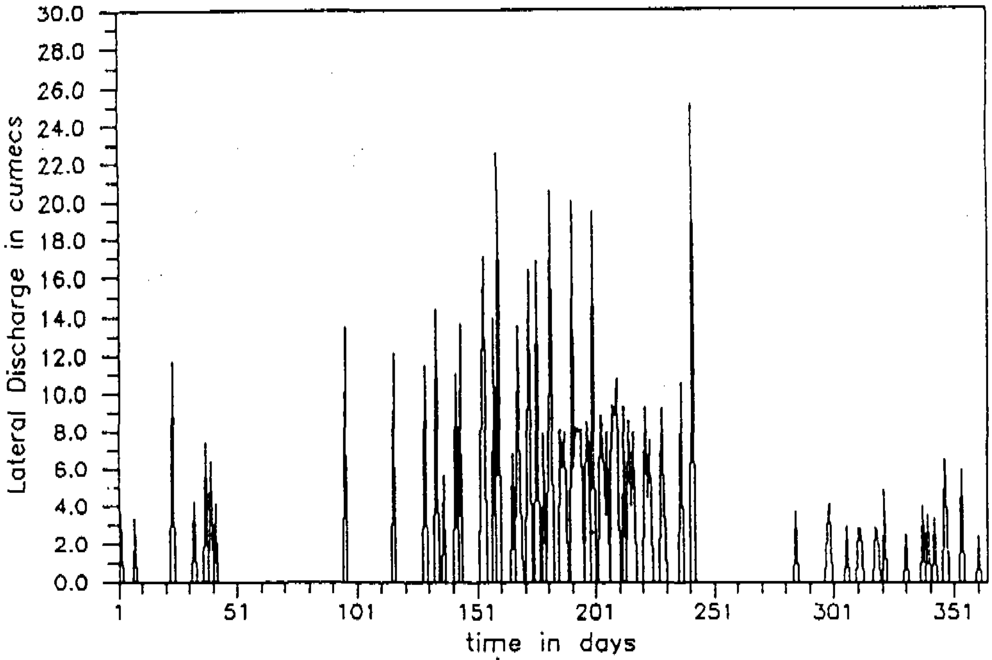


Fig. 4.a Simulated Lateral Demand for the Lateral L1 in El-Nasr Canal by using the Irrigation Scheduling Criteria to Irrigate at Stress Days Only, to Raise the Soil Moisture upto Field Capacity.

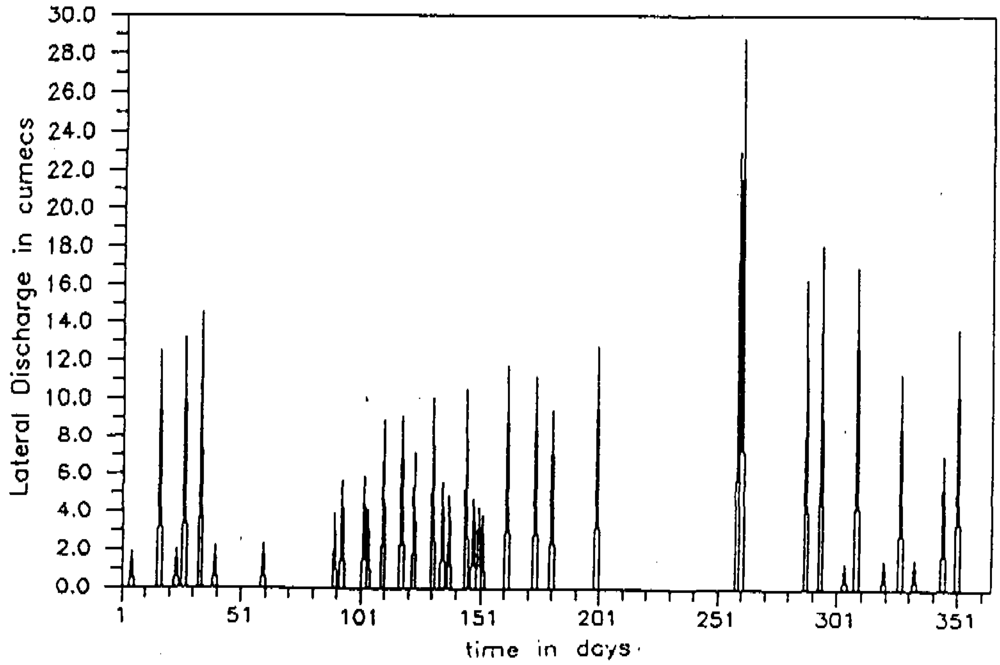


Fig. 4.b Simulated Lateral Demand for the Lateral L1 in Hypothetical Canal, by using the Irrigation Scheduling Criteria to Irrigate at Stress Days Only, to Raise the Soil Moisture upto Field Capacity.

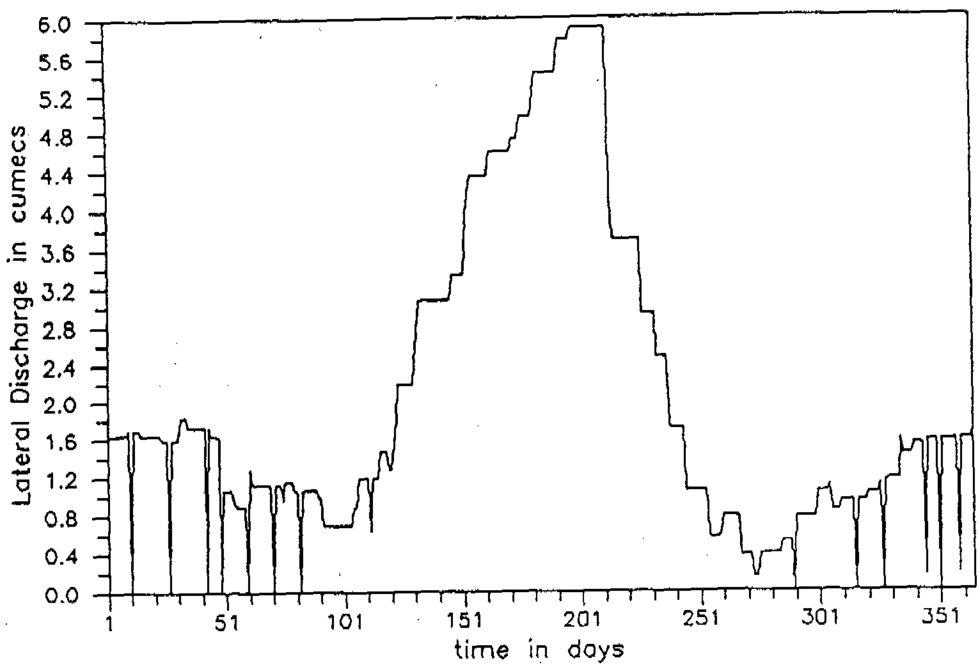


Fig. 5.a Simulated Lateral Demand for the Lateral L1 in El-Nasr Canal by using the Irrigation Scheduling Criteria to Maintain Soil Moisture at Field Capacity Limit.

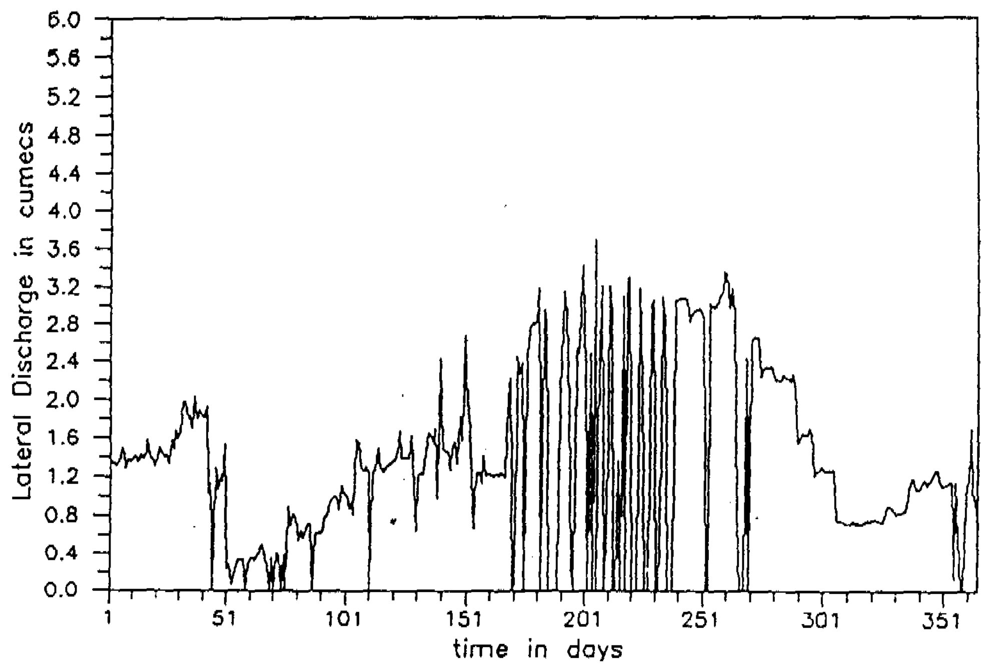


Fig. 5.b Simulated Lateral Demand for the Lateral L1 in the Hypothetical Canal, by using the Irrigation Scheduling Criteria to Maintain Soil Moisture at Field Capacity Limit.

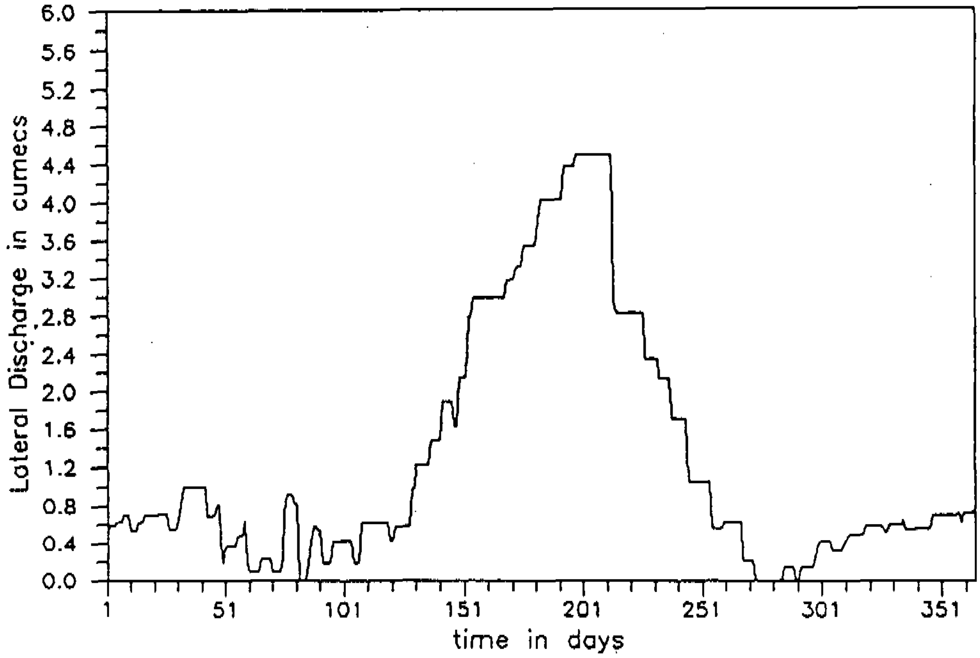


Fig. 6.a Simulated Lateral Demand for the Lateral L1 in El-Nasr Canal by using the Irrigation Scheduling Criteria to Irrigate at Stress Days Only, with SWR sufficient for One Day Crop Water Requirement.

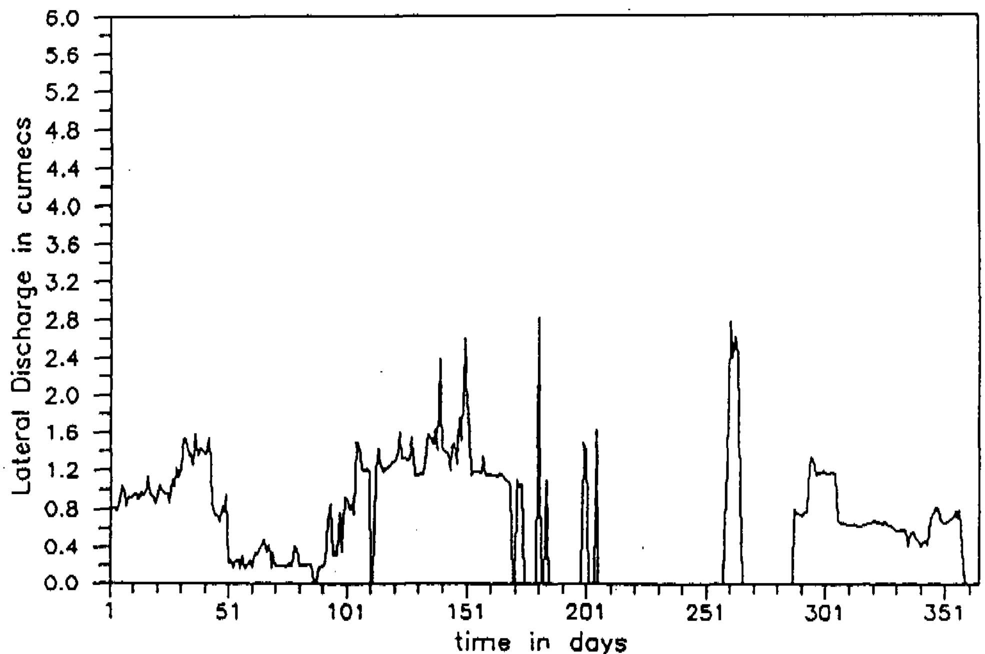


Fig. 6.b Simulated Lateral Demand for the Lateral L1 in the Hypothetical Canal, by using the Irrigation Scheduling Criteria to Irrigate at Stress Days Only, with SWR sufficient for One Day Crop Water Requirement.

limit) by using the supplementary irrigation water. Therefore, this scheduling criteria is also not recommended because it provides very poor rainfall utilization and increased water requirements.

6.3 Irrigation Scheduling Criteria - 3

To obtain maximum effective rainfall, supplementary irrigation requirements are computed based on maintaining the soil moisture slightly above the depletion limit and below the field capacity. This approach maintains more air space within the soil to be used by any probable rainfall. Amount of supplementary irrigation is computed to be sufficient to meet one day crop water requirement only, without stress. The result is a high reduction in lateral demand as shown in Figure 6.

7. CONCLUSION

Based on the above results, it can be concluded that the proposed DDSM is capable on simulating the daily lateral canal discharge requirements with different irrigation scheduling criterion. Considerable amounts of irrigation water can be saved and the need for high design capacity of the canal can be avoided through better selection of the irrigation scheduling criteria. The irrigation scheduling criteria in which the soil moisture content is maintained slightly above the wilting point is the best when drip or sprinkler irrigation systems are used.

Real-time operation of the canal can be based on the actual current meteorological data and rainfall forecasts in the fields. The proposed 'DDSM' in this paper has its potential usefulness for on-line automatic regulation of canal systems.

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PLANIFICATION D'UN RÉSEAU MÉTÉOROLOIQUE APPLICATION À LA MAURITANIE

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Résumé

L'étude du réseau météorologique de la Mauritanie, a été réalisée, suivant une approche régionale en faisant appel à l'Analyse en Composantes Principales (ACP) et à l'interpolation optimale. Les notions théoriques particulières à chacune de ces deux méthodes d'analyse sont présentées. On utilise les données annuelles des précipitations de 24 stations (1940 - 1990) en Mauritanie. L'application de l'ACP de type Varimax avec rotation orthogonale des composantes de la matrice des données A = I x J où I = 51 (nombre d'années) et J = 24, le nombre de stations, fait ressortir les régions pluviométriques homogènes qui correspondent aux premières composantes principales significatives. On explique, par la suite, avec exemples à l'appui, dans le cadre de la planification du réseau météorologique de la Mauritanie, l'estimation des fonctions de structure et des écarts types d'interpolation régionaux, en fonction de la distance entre les stations, et la cartographie des écarts types réels du réseau en opération. On indique, aussi comment les modifications projetées à ce réseau peuvent être évaluées.

Abstract

A study of Mauritanian meteorological network was performed by means of regional approach according to Principal Component Analysis (PCA) and optimal interpolation. The yearly precipitation values of 24 stations meteorological stations (1940 - 1990) of Mauritania are used. The application of ACP, varimax type with orthogonal rotation of components of the data matrix A = I x J, where I = 51 (the number of the years) and J = 24, the number of the station reveal three homogenous precipitation regions, which correspond to the three first significant principal components. The specific theory for each of these tow methods is briefly reviewed. This is followed by an explanation, with examples, of the way in which these two methods are applied , in connection with the planification of the Mauritania meteorological network, in order to determine the statistically homogenous region, to estimate the regional structure functions and standard errors of interpolation as functions of the distance between stations, and to map the calcul standard errors of the interpolation for the network being operated at a given date. It is also explained how modifications forseen for this network can be evaluated.

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1. INTRODUCTION

Face à l'évolution de ses réseaux et des besoins en données météorologiques, le service de météorologie en Mauritanie a jugé qu'il devenait nécessaire d'analyser plus en détail son réseau d'acquisition de données météorologiques en vue de planifier plus adéquatement son développement futur. C'est dans ce cadre qu'on a voulu réaliser une étude de planification portant sur le réseau d'acquisition de données pluviométriques de la Mauritanie.

Les objectifs généraux définis pour l'étude visent à cerner les besoins à satisfaire, puis à étudier dans quelle mesure le réseau étudié peut déjà ou pourra fournir les données désirées, compte tenu des contraintes financières, humaines, techniques et logistiques inhérentes à la création et à l'opération de nouvelles stations. Dans le but de définir le type de rationalisation le plus adéquat pour réaliser les objectifs fixés, on a procedé tout d'abord à une revue approfondie de la littérature, qui a permis de considérer des aspects forts variés de la planification des réseaux et d'établir les choix possibles.

2. LE RÉSEAU PLUVIOMÉTRIQUE ACTUEL DE LA MAURITANIE

2.1 Pluie Journalière

Le réseau comporte, d'après un tableau fourni par le service agrométéorologique de la direction de l'agriculture (SAMA), 81 stations de mesure dont 13 stations synoptiques gérés par l'ASECNA (Agence de Sécurité de la Navigation Aérienne), les autres postes pluviométriques et stations agrométéorologiques dépendent du SAMA. D'autres postes pluviométriques existent certainement en dehors de ceux officiellement répertoriés, postes installés à l'occasion de projets ou d'études de plus ou moins longues durées. Près de 80% de ces stations sont situées au Sud du pays du 17° parallèle Nord (voir figure 1), avec une très forte concentration dans les régions du Sud du pays. Seulement 13 stations contrôlent la partie du pays située au Nord du 18éme parallèle Nord. Sur ces 81 stations notons que 22 ont une période d'observation supérieure ou égale à 30 ans, 25 dépassent ou atteignent les 20 ans d'observation, 31 sont observées depuis au moins 10 ans, 65 depuis au moins 5 ans. Beaucoup de ces stations ont été créées dans les années 1978 - 1980, dans le Brakna*, l'Assaba*, le Guidimaka*, et le Gorgol*. Les relevés de onze d'entre elles n'ont pas été communiquées par le SAMH (Service d'Agrométéorologie de la direction d'Hydraulique). Un nombre important de stations présentent des lacunes d'observation de plus ou moins longue durée (voir figure 2). C'est ainsi que pour l'étude de la pluviométrie annuelle ou mensuelle sur la période 1940 - 1990, soit 51 ans, nous n'avons pu utiliser que 24 stations alors que 30 dépassent théoriquement les 60 années d'observation.

2.2 Pluviographie

Le SAMH ne possède actullement pas de pluviographes en fonctionnement. Il ne semble pas que du côté de l'ASECNA, la situation soit meilleure. Si des renseignements existent, ils n'ont pas été exploités ou tout au moins les résultats de cette éventuelle exploitation ne sont pas disponibles au SAMH. Jusqu'à plus ample information, les seules données utilisables pour des études d'intensité en Mauritanie, sont les courbes Intensité - durée - fréquence, publiées par Y.Brunet Moret en 1965.

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* des régions du Sud de la Mauritanie.

3. MÉTHODES ANALYTIQUES

Il n'est pas inutile de rappeler ici que l'objectif poursuivi par la collecte des données pluviométriques à un certain nombre de stations formant un réseau, est l'estimation des valeurs prises par une variable météorologique en tout point du champ considéré. D'où la nécessité d'interpoler les valeurs observées aux stations. On est dès lors amené à se demander quelle est la précision de cette interpolation. Plusieurs méthodes d'interpolation ont été suggérées et appliquées jusqu'à maintenant, mais si toutes permettent d'estimer une valeur prise par une variable en un point donné, toutes ne permettent pas d'estimer aussi l'erreur d'interpolation commise. C'est toutefois précisement ce qu'il faut pour analyser dans quelle mesure un réseau permet de répondre aux objectifs fixés et pour déterminer les modifications susceptibles d'améliorer la précision de ce réseau.

L'objectif de cet article est donc de présenter les méthodes d'analyse qui ont été retenues dans le cadre de l'étude du réseau pluviométrique de la Mauritanie, pour l'estimation des valeurs ponctuelles dans une région donnée. Suite à une revue élaborée de la littérature (Morin et al, 1980) la méthode d'interpolation optimale développée par Gandin (1965), servant déjà à certains systèmes de prévisions numériques, a été retenue pour réaliser les analyses du réseau prévu. Comme toute méthode de calcul, l'interpolation optimale repose sur des hypothèses. En particulier, les calculs sont simplifiés de beaucoup si le champ étudié s'avère homogène et isotrope par rapport à la fonction employée. Idéalement donc, l'interpolation optimale doit être appliquée à des champs homogène et isotropes.

Compte tenu des dimensions de la Mauritanie et des variations de physiographie rencontrées, il a paru évident dès le début que les données recueillies par le réseau pluviométrique mauritanien n'étaient pas homogène spatialement. D'où le désir de travailler non pas à la grandeur du pays, mais de le diviser en régions à l'intérieur desquelles les données pourraient être considérées comme homogène, cette division en régions ayant l'avantage espéré de mieux faire ressortir les caractéristiques régionales qui auraient risqué d'être masquées dans une étude globale de l'ensemble du réseau.Définir des régions à l'intérieur desquelles les stations sont homogènes par rapport à celles qui sont situées à l'extérieur, est effectivement possible. On peut tenter de le faire plus ou moins arbitrairement, en se basant, par exemple, sur une carte de pluies totales annuelles, ou sur la physiographie.

Il existe toutefois une méthode d'analyse des données dont nous n'avons pas parlé jusqu'ici explicitement et qui peut solutionner ce problème de façon relativement satisfaisante: l'analyse en composantes principales (Dyer, 1975). Par ailleurs, Morin et al., (1978 et 1979) ont vérifié avec succés la possibilité d'utiliser ce type d'analyse afin de définir des groupes de stations homogènes pour l'application de l'interpolation optimale. On démontre dans les paragraphes qui suivent comment ces deux méthodes se complètent pour fournir une analyse des caractéristiques statistiques régionales du réseau pluviométriques de la Mauritanie. Quant à l'anisotropie, il est possible d'en tenir compte en calcualnt les fonctions désirées selon l'orientation du segment joignant deux stations. On obtient alors des valeurs différentes des paramètres des fonctions, selon l'orientation choisie. En pratique, on choisit deux classes d'orientations, l'une parallèle à l'orientation du vent dominant et l'autre perpendiculaire. Dans le cadre de l'étude du réseau météorologique de la Mauritanie, on suppose que l'hypothèse d'isotropie s'applique au moins de façon approximative.

A: ANALYSE EN COMPOSANTES PRINCIPALES.

A.1. Objectif

L'objectif essentiel de l'analyse des données en composantes principales, dans le cadre de l'étude du réseau météorologique de la Mauritanie, est de déterminer des groupes de stations relativement homogènes, caractérisant des "régions météorologiques statistiquement homogènes", dont la structure statistique sera étudiée plus en détails par interpolation optimale, dans le but de planifier le réseau régional.

A.2. Théorie

Ce n'est pas notre intention de rappeler ici en détails la théorie de l'analyse en composantes principales; ces renseignements peuvent facilement être obtenus dans la littérature (Hotelling, 1933; Kendall, 1957; Anderson, 1958; Matalas et Reither, 1967). Nous nous limiterons à une brève description des caractéristiques principales de cette théorie. Un groupe de p variables, contenant chacune n observations, peut être représenté par une matrice de dimensions p x n. Chaque observation peut aussi être située dans un espace à p dimensions. Cependant, comme les variables peuvent être corrélées, les p axes de cet espace ne sont pas toujours orthogonaux. L'objectif de l'analyse en composantes principales est de transformer les variables de telle sorte que les axes deviennent orthogonaux, ce qui permet alors la définition de nouvelles variables qui, elles, sont indépendantes.Une fois celà fait, on peut déterminer les coefficients de corrélation r_{jk} entre la station (ou variable) j et la composante (ou axe) principale d'ordre k:

$$r_{jk} = q_{jk}\, l_{k} \qquad (1)$$

q_{jk} est le vecteur propre entre la station j et la composante principale k et l_{k} est la valeur propre de la composante k.

L'analyse de la variation de r_{jk} fournit des informations précieuses sur le comportement relatif des stations. Il pourrait se trouver que les r_{jk} soient identiques pour deux ou plusieurs stations, ce qui impliquerait des corrélations parfaites entre ces stations. C'est toutefois rarement le cas, par suite d'erreurs d'observations et d'irrigularités microclimatiques (Gandin, 1970). La solution est alors de supposer que les stations dont les coefficients de corrélation avec les q premières composantes principales sont relativement similaires, sont statistiquement homogènes et peuvent être regroupées ensemble. Le nombre de composantes à considérer n'est pas défini de façon rigoureuse. En général, ce choix est basé sur le pourcentage de la variance totale expliqué, après considérations des q premières composantes.

Une façon de définir ces groupes plus objectivement consiste à faire subir une rotation aux axes principaux, de manière à redistribuer le pourcentage total de variance expliquée plus également entre les q composantes (axes). C'est la méthode VARIMAX qui a été retenue pour ce faire (Nie et al. , 1975). Comme précédemment, les coefficients de corrélation entre les nouveaux axes principaux et les stations peuvent être calculés.

Suite à cette rotation, pour chaque station, on identifie le coefficient de corrélation le plus élevé entre cette station et l'un des axes. Les stations sont alors regroupées selon l'axe avec lequel leur corrélation est la plus élevée, formant autant de groupes qu'il y a d'axes (Dyer, 1975). On peut, de plus, détecter des sous-groupes par l'analyse des coefficients de corrélation entre les stations et les autres axes.

A.3. Application au réseau météorologique de la Mauritanie pour la définition des "régions statistiquement homogènes".

Pour appliquer la méthode des composantes principales, qui utilise les corrélations entre les stations prises deux à deux, il est nécessaire de s'assurer une période d'observation concomitante suffisamment longue pour que les coefficients de corrélation soient significatifs. Dans notre cas on a choisi d'utiliser les 24 stations pour lesquelles les données sont disponibles sur 51 ans. On a choisi d'étudier le réseau pour les précipitations annuelles. Nous avons appliqué l'ACP de type Varimax avec rotation orthogonale des composantes aux données mensuelles et annuelles mentionnées ci-dessus. La matrice des données est la suivante: A = I x J = 24 X 51 et c'est l'information continue dans cette matrice que nous allons étudier.

Le tableau 1 comprend les valeurs propres ainsi que les pourcentages des six premières composantes significatives. Ce tableau nous montre que la première composante explique 38 % de la variance et qu'avec les six premières composantes on explique 79 % de la variance totale des observations, ce qui est un indice d'homogénéité du réseau étudié (24 stations) pour les précipitations annuelles. Si l'on compare les coefficients de corrélation des stations avec la première composante principale on voit que l'influence de cette composante n'est pas la même pour toutes les stations. Les valeurs des coefficients de corrélation ou de variance expliquée varient d'une composante à l'autre et dépendent des observations. Si le coefficient de corrélation entre deux stations est égal à 1 (sans que les stations aient nécessairement une même moyenne ou un écart type) les coefficients de corrélation entre les composantes et ces deux stations seront égaux. Comme on observe rarement des corrélations parfaites même pour deux stations météorologiques voisines, par suite d'erreurs de mesure et de conditions micro-climatiques locales, on doit rechercher les groupes homogènes en associant les stations de comportement semblable sur l'ensemble des premières composantes.

Le nombre de composantes à considérer ne suit pas de règle rigide. En général, on tient compte de la variance expliquée par la composante et de la variance totale expliquée. dans ce cas-ci on considère les six premières composantes. Il existe deux méthodes simples de présenter le tableau 1 sous forme de figure de manière à déterminer les groupes de stations dont le comportement est semblable. La première est la présentation dans l'espace à trois dimensions des corrélations entre les stations et les trois premières composantes. La deuxième méthode, qui est moins employée mais plus discriminante et qui contient plus d'informations à notre point de vue, est une projection dans le plan des composantes 2 et 3, des corrélations avec les autres composantes. Contrairement à la première méthode cette deuxième méthode peut présenter plus de trois composantes simultanément. Les résultats pour la période "annuelle" sont présentés et discutés avec les deux formes de graphiques pour qu'on puisse se rendre compte des avantages de l'une sur l'autre.

La figure 3 est la présentation en trois dimensions du tableau 1. Les axes R1, R2, R3 représentent respectivement les composantes 1, 2, 3. L'origine des segments de droite qui représentent chacun une station a pour coordonnées les corrélations de la station avec les composantes deux et trois, tandis que la longueur du segment de droite représente la corrélation de la station avec la première composante. Les stations peuvent être regroupées (voir figure 3). La figure 4 explique comment on met en graphique les données du tableau 1 selon la seconde méthode. Comme dans le cas précédent, on détermine tout d'abord le point A d'un segment de droite $\overrightarrow{AB}$ en prenant comme coordonnées sur les axes R2 et R3 (plan des composantes 2 et 3) les valeurs respectives des coefficients de corrélation entre la station et les composantes deux et trois. La longueur du segment $\overrightarrow{AB}$ est égale à la valeur prise par le coefficient de corrélation entre la station et la première composante principale. Ce segment $\overrightarrow{AB}$ est construit dans le plan des axes R2 et R3 perpendiculairement au segment $\overrightarrow{OA}$ (orthogonalité des composantes principales). Enfin, si par convention, la corrélation est positive on trace le segment $\overrightarrow{AB}$ dans le sens des aiguilles d'une montre et dans le sens contraire si elle est négative. Le segment $\overrightarrow{OB}$ est le coefficient de corrélation multiple entre la station et les six premières composantes. On peut tracer ainsi toutes les compoantes. Cependant compte tenu du peu de variance expliquée par les dernières composantes on trace rarement plus de 6 composantes.

La figure 5 permet de trouver les groupes de stations par la proximité des segments et leur pente. Elle nous montre également l'importance des premières composantes et le nombre minimum utile pour la reconstitution des observations à chaque station. Rappelons en effet que l'on peut reconstituer les observations pour chaque station à l'aide des K premières composantes. Cette reconstitution des observations sera d'autant meilleure que le coefficient de corrélation multiple s'approchera de 1, ce qui nous est indiqué sur le graphique par la proximité de l'extrémité de la ligne relative à la station avec le cercle qui correspond à une corrélation de 1.L'étude de la figure 5 nous montre qu'avec 6 composantes les observations de la plus part des stations peuvent être bien reproduites (proximité de la fin de chaque segment avec le cercle). On groupe comme précédement les stations (voir fig 5). Les groupes de stations ont été identifiés sur les graphiques par des points, ou segments de droite, voisins indiquent des contributions identiques sur le calcul des composantes. On peut, si on le désire, faire ressortir des groupes en faisant subir une rotation aux premiers axes principaux.

Pour les précipitations annuelles, par exemple si l'on fait cette rotation (on utilise la méthode Varimax*) avec les 6 premiers axes principaux, la variance expliquée sur l'ensemble demeure 79% mais est distribuée différement sur les 6 axes. La rotation étant faite on détermine les groupes en faisant ressortir pour chaque station l'axe de contribution maximum. Le tableau 2 montre les valeurs obtenues après rotation et les stations soulignées sur une même colonne forment un groupe. La rotation des axes est faite après avoir imposé le nombre d'axe que l'on désire garder. Dans ce cas-ci on a imposé la rotation en ne conservant que les six premiers axes. Il est évident que si l'on impose par exemple la rotation avec 7 axes, quelques stations à la limite d'un groupe peuvent former un autre groupe. Par exemple la contribution de la station Akjoujt sur l'axe 4 n'est peut être pas significativement différente de sa contribution à l'axe 2 et pourrait à la rigueur faire partie du groupe de l'axe 2 ou d'un septième groupe.

A.4. Interprétation des résultats

Rappelons que les stations situées dans les régions ainsi définies sont regroupées selon la composante (après rotation) avec laquelle leur coefficient de corrélation est le plus élevé. Il est possible, d'autre part, pour chaque composante principale, de tracer la variation spatiale des coefficients de corrélation entre cette composante et les différentes stations . Le choix des composantes significatives a été fait par l'application de la régle de Kaiser (1959) selon laquelle les valeurs propres sont supérieures ou égale à 1.

Pour la meilleure interprétation des résultats obtenus par l'ACP on a tracé sur la carte gégraphique de la Mauritanie les isolignes en utilisant les valeurs des coefficients de corrélation des variables (stations) avec les trois premières composantes principales significatives. La figure 6 est le fruit de ce travail tandis que la figure 7 est une synthèse de la figure 6. Dans le Sud de la Mauritanie, où le nombre de stations est relativement élevé, les "limites" des régions sont relativement stables, lorsque l'on compare les cartes initiales. Cependant, il faut comprendre que ces régions ne font qu'exprimer une influence prépondérante d'une composante principale. En d'aures mots, une station située près des limites d'une région est reliée de façon prépondérante à une composante principale, mais l'est aussi à un degré moindre avec les composantes caractérisant les régions environnantes. Surtout dans le Nord et le Nord-Est du pays, le nombre restreint de stations empêche de définir les régions de façon vraiment satisfaisante. On ne peut alors compter que sur quelques stations pour ce faire. Les limites deviennent alors plus imprécises et il est même possible d'ignorer des régions, faute de données, surtout si ces régions sont situées en plein désert.

L'ensemble des résultats obtenus pour le réseau pluviométrique étudié au pas de temps annuel, indique que les dimensions des "régions" définies à l'aide des composantes principales dépendent beaucoup plus de caractéristiques saisonnières des masses d'air et des perturbations atmosphériques qui survolent la Mauritanie que de sa physiographie. Par ailleurs, la constatation que lorsqu'il pleut à la station n1, il pleut aussi à la station n2 n'implique pas nécessairement que la moyenne et la variance à ces deux stations soient homogènes. L'homogéneité d'un groupe de stations représentatives d'une région se situe davantage au niveau de leur comportement similaire, exprimé par leur coefficient de corrélation avec la composante principale caractéristique de la région qu'au niveau de leur moyenne et de leur variance, ce qui n'exclue pas que ces dernières puissent être relativement homogènes.

B: INTERPOLATION OPTIMALE

B.1. Objectif

Dans le cadre de l'étude du réseau météorologique de la Mauritanie en vue de sa planification, l'objectif visé, en appliquant la méthode de l'interpolation optimale pour l'analyse des données disponibles pour chaque "région météorologique statistiquement homogène", est d'obtenir les informations nécessaires à la détermination de la configuration optimale d'un réseau apte à répondre à des besoins en données météorologiques exprimés sous forme de précision souhaitée. Cette analyse des données, par interpolation optimale, a donc pour objectifs spécifiques de déterminer:

- l'écart type d'interpolation en fonction de la distance entre les stations, dans le cas d'un réseau régulier;

- l'écart type réel d'interpolation en tout point d'une région, obtenu à partir du réseau en place, à une date donnée.

B.2. Structure des champs météorologiques

Compte tenu des phénomènes d'échelle planétaire ou hémisphérique qui sont à l'origine des variations météorologiqus à la surface de la terre, les variables météorlogiques (pluie, température,...) présentent une certaine structure spatiale et temporelle. D'autre part, comme on l'a rappelé précédemment, à cette structure supra-régionale ou régionale, se superposent, particulièrement dans le cas des champs de surface, des variations d'échelle beaucoup plus courtes, induites par l'hétérogéneité de la surface du sol (relief, végétation,...).

Gandin (1965) précise que ces variations affectent l'homogéneité des normales de ces variables, de sorte que les fonctions de structure des valeurs observées de variables météorologiques ne sont en général ni homogènes ni isotropes. Ces fonctions FS dépendent alors de la position des stations i et j où les observations fi et fj des variables sont effectuées. Elles peuvent s'écrire:

$$\mathbf{F.S.} = \frac{\sum_{i=1}^{m} (f'_{ij} - f'_{ik})^2}{m} \qquad (2)$$

où

f'_{ij} est la Iième déviation de la station j;

f'_{ik} est la Iième déviation de la station k;

f' étant la déviation de l'observation par rapport à la moyenne des observations de la station;

m est le nombre d'observations.

Les valeurs obtenues de l'équation (2) sont utilisées pour définir la fonction de structure du phénomène étudié en fonction de la distance qui sépare chaque couple de stations. En pratique l'axe des distances est subdivisé en classes et on calcule à l'intérieur de chaque classe la valeur moyenne de la fonction de structure. L'ajustement de la courbe passant par ces valeurs en fonction de la distance nous permet par la suite d'interpoler la fonction de structure (ou fonction de covariance) entre les stations et le point à interpoler à l'aide des distances de ce point à chacune des stations. De plus en extrapolant la courbe à une distance zéro on peut estimer l'erreur minimale ponctuelle à laquelle on peut s'attendre. Cette erreur est dûe à la fois à l'imprécision des mesures et à l'influence du micro-climat. La mise en graphique des valeurs FS(d) calculées pour chaque couple de stations, à partir des séries chronologiques observées à ces stations, présente l'aspect d'un nuage de points plus ou moins serré, comme on peut le constater à la figure 8a. La dispersion de ces points traduit graphiquement dans quelle mesure l'hypothèse d'homogénéité, en particulier, est respectée ou non. Plus le nuage est étroit, plus le champ est homogène; plus il est large, plus le champ est hétérogène.

Par ailleurs, un nombre restreint de points se détachant nettement du nuage est généralement l'indice d'une station non représentative, au niveau régional. Il s'agit habituellement d'une station limitrophe, assez fortement corrélée avec les stations d'une région adjacente, ou encore, d'une station influencée par des caractéristiques physiographiques locales particulières.

B.3. Théorie générale

Pour estimer les précipitations en un point on doit déterminer le poids à donner à chacune des stations. Ces poids sont déterminés en minimisant l'erreur quadratique moyenne, ce qui nous donne un système à N équations à résoudre lorsqu'on a N stations. Soit à résoudre:

$$\sum_{j=1}^{N} s_{ij}^2 p_j^2 = s_{0i}^2 \quad (i = 1, 2, ..., N) \qquad (3)$$

où

s_{ij}^2 sont les covariances entre les stations i et j;

s_{0i}^2 sont les covariances entre le point à extrapoler et les stations;

p_j^2 sont les poids à donner à chacune des stations.

Ce système est régulier et admet une seule solution qui donne les N poids Pj qui minimisent la variance. Les valeurs et sont prises sur la fonction de structure en fonction des distances inter-stations et des distances entre le point à estimer et les stations. Connaissant les poids, on peut calculer la variance de l'estimation par:

$$E_0^2 = \sum_{i=1}^{N} P_i s_{0i}^2 \qquad (4)$$

où

E_0^2 = variance de l'estimé au point 0;

s_{0i}^2 = variance entre le point et chacune des stations, prises sur la fonction de structure en fonction de la distance.

L'estimé du phénomène au point nous est donné par:

$$V_0 = \sum_{i=1}^{N} P_i f'_i + \overline{f_0} \qquad (5)$$

où

V_0 = valeur au point zéro;

P_i = poids de chacune des stations;

f'_i = déviation de l'observation de la station i par rapport à la moyenne des observations de cette station;

$\overline{f_0}$ = norme ou moyenne au point zéro

Si l'on connaît pas la norme à tous les points, on peut faire les calculs directement avec les observations en imposant une contrainte supplémentaire ($\sum P_i = 1$) pour que l'estimé soit sans biais. C'est cette méthode que nous utilisons. Le système d'équations (3) devient un système à N + 1 équations, le N + Iième paramètre est le multiplicateur de Lagrange. En pratique, pour tracer les iso-valeurs ou les iso-erreurs on fait les calculs sur un grand nombre de points répartis sur la région.On peut cependant dire que les iso-valeurs calculées par cette méthode sont plus précises que par les méthodes citées précédemment parcequ'elle tient compte de la structure spatiale du phénomène et de l'erreur de mesure et de micro-climat.

B.4. Écart type d'interpolation en fonction de la distance entre les stations

La géométrie d'un réseau réel est le plus souvent tributaire de la physiographie, des axes routiers et des centres de population. Il est toutefois important, comme point de comparaison, de déterminer qu'elle serait l'écart type d'interpolation obtenue à partir d'un réseau de géométrie régulière. Gandin (1965 et 1970) indique qu'un réseau à mailles triangulaires est plus susceptible d'être réalisé de façon approximative qu'un réseau à mailles carrées. Le même auteur suggère donc de calculer l'écart type d'interpolation au centre d'un triangle équilatéral aux sommets duquel les stations sont situées. Toutefois, les considérations exprimées en sur le nombre de stations souhaitables dans un voisinage et leur disposition autour du point de grille, de même que les essais de calculs que nous avons réalisées, indiquent qu'il est préférable de prendre six stations. Nous avons opté pour un réseau de six stations disposées aux sommets du triangle (figure 9). La distance entre les stations du triangle est l et celle entre les stations du triangle extérieur est 2l. Six stations situées tout autour du point de grille sont donc retenues. Nous avons constaté que l'addition d'une autre couronne de stations n'améliorait pas l'écart type de façon importante et avons décidé de nous en tenir à six stations de façon à compenser pour l'impossibilité pratique d'obtenir un réseau de géométrie régulière et par conséquent d'obtenir des écarts types aussi faibles.

Dans le cas particulier du calcul de l'écart type d'interpolation au centre de deux triangles équilatéraux emboîtés, les poids p attribués aux trois stations intérieures sont identiques. De même, les poids q attribués aux stations extérieures sont aussi identiques entre eux, mais différents des poids p. Si le système d'équations est solutionné pour ce cas, alors:

$$P = \frac{b\left(\frac{2l}{\sqrt{3}}\right) - \tilde{b}\left(\frac{l}{\sqrt{3}}\right) + \frac{1}{3}\left[2\tilde{b}(l) + \tilde{b}(l\sqrt{3}) - 2\tilde{b}(2l)\right]}{2\left[\tilde{b}(l) + \tilde{b}(l\sqrt{3}) - \tilde{b}(2l)\right]} \qquad (6)$$

$$q = \frac{1}{3} - p \qquad (7)$$

$$l = \tilde{b}\left(\frac{l}{\sqrt{3}}\right) - \frac{1}{3}\left[\tilde{b}(l\sqrt{3}) + 2\,\tilde{b}(l)\right] + \tilde{b}(l\sqrt{3})\,p \qquad (8)$$

Comme on peut le constater à la figure 9, la distance l est la distance de base du réseau et correspond à la distance qui sépare une station de ses voisines immédiates. Les valeurs des fonctions de structure expérimentales $\widetilde{b_{ij}}$ entre les diverses stations doivent être évaluées pour des distances égales à l, 2l et $l\sqrt{3}$, tandis que les $\widetilde{b_{0i}}$, entre le point de grille et les stations correspondent à des distances de $\frac{l}{\sqrt{3}}$ et $\frac{2l}{\sqrt{3}}$ L'équation 1.1 devient alors:

$$E = 3\left[p\,\tilde{b}\left(\frac{l}{\sqrt{3}}\right) + q\,\tilde{b}\left(\frac{2l}{\sqrt{3}}\right)\right] + l - s^2 \qquad (9)$$

Comme précédemment, s^2 est l'ordonnée à l'origine de la fonction de structure expérimentale $\widetilde{b_{ij}}$. En estimant p, q et l par rapport aux valeurs de la fonction de structure expérimentale $\widetilde{b_{ij}}$, dont l'argument est exprimé sous forme de distance, il est donc possible, à l'aide de l'équation (9) d'exprimer l'erreur quadratique moyenne ou l'écart type moyen en fonction de la distance entre les stations d'un réseau à mailles triangulaires (figure 10). Un point important doit être souligné ici. La variation de l'écart type en fonction de la distance l entre les stations n'est pas une simple transposition de la valeur prise par la fonction de structure à la même distance et donc de la forme de la fonction de structure. Tel qu'indiqué plus haut, l'écart type à une distance l donnée résulte de relations mathématiques utilisant les valeurs de la fonction de structure à cinq (5) distances différentes (l, 2l, $\frac{l}{\sqrt{3}}$, $l\sqrt{3}$ et $\frac{2l}{\sqrt{3}}$) et non seulement à la distance l.

Il est à noter que cette courbe présente l'écart type moyen dans une région homogène et isotrope. Les écarts types obtenus pour une distance donnée sont donc à interpréter à la lumière des informations disponibles sur le degré de réalisation de ces propriétés du champ dans la région étudiée. L'étude de cette courbe permet aussi de déterminer la distance requise entre les stations pour atteindre une précision donnée et par suite d'évaluer le réalisme d'un tel critère, en fonction du nombre de stations que celà implique et des coûts associés. On peut aussi constater, par exemple, que l'erreur ne diminue pas tellement même si la distance entre les stations diminue de façon appréciable et que ce n'est que si les stations sont relativement près les unes des autres que l'erreur commence à vraiment diminuer. L'ordonnée à l'origine est aussi significative. Elle dénote qu'il est impossible d'obtenir, en moyenne, une erreur plus faible, quelle que soit la distance entre les stations. Naturellement, tout comme les fonctions de structure changent de région en région et avec le mois, cette erreur minimale fluctue dans l'espace et dans le temps (mois).

B.5. Cartographie des écarts types d'interpolation du réseau actuel

Ce n'est pas suffisant de connaître quel serait l'écart type d'interpolation d'un réseau de géométrie régulière dont la distance entre les stations est d. Il importe de vérifier dans quelle mesure le réseau en opération à une date donnée permet d'atteindre la précision désirée. Un réseau réel possède, en effet, une géométrie plus ou moins irrégulière. Il peut donc arriver que, dans certaines parties d'une région, la densité du réseau soit amplement suffisante pour atteindre et même dépasser la précision souhaitée, alors qu'ailleurs, dans la même région, la densité et la gémétrie du réseau soient telles que la précision souhaitée est loin d'être atteinte. La cartographie des écarts types d'interpolation permet donc de vérifier l'aptitude d'un réseau à répondre à des critères de précision déterminés, en tout point d'une région et, par suite, de modifier ce réseau en conséquence. En pratique, l'écart type d'interpolation à partir du réseau existant est calculé en chaque point d'une grille carrée, en résolvant les équations (8) et (9) pour chaque point de grille et les iso-écarts types sont tracés par ordinateur, en interpolant les valeurs obtenues aux point de grille. La figure 11 présente une telle carte, basée sur les stations en opération et situées à l'intérieur de la zone d'influence d'une région donnée.

L'analyse des écarts types doit porter principalement sur la distribution des écarts types à l'intérieur de la limite de la région. Le calcul a toutefois été étendu jusqu'aux limites de la zone d'influence, en considérant, en particulier, que les caractéristiques propres à une région ne s'arrêtent pas brusquement à sa limite, mais diminuent plus ou moins rapidement dans les régions contigue. Ces valeurs peuvent donc çetre utilisées pour modifier, à la hausse ou à la baisse, les écarts types estimés, dans chacune des régions contigues sur lesquelles la zone d'influence de la région étudiée s'étend.

B.6. Interprétation des résultats

Il importe tout d'abord de noter que les écarts types présentés aux figures 10 et 11 sont des valeurs moyennes et que pour un événement donné, l'écart réel d'une interpolation basée sur les observations aux points de mesure peut s'avérer fort différent de cet écart type moyen. Il faut donc interpréter ces valeurs dans un sens statistique.

L'examen d'une carte d'iso-écarts types permet rapidement de localiser les parties d'une région où les écarts types sont les plus faibles, de même que celles où ils sont les plus élevés. Logiquement, toute partie d'une région pour laquelle l'écart type d'interpolation est élevé, est sujette à une augmentation de la densité de stations. Il faut alors vérifier si cette valeur élevée est supérieur à l'écart type accepté. Si oui, la courbe d'écarts types en fonction de la distance (voir fig. 10), réalisée pour le même pas de temps et la même région, permet de déterminer la distance d requise entre les stations pour diminuer l'écart type et atteindre la précision requise. Si la distance requise est trop faible et irréaliste, il y a lieu de reviser le critère de précision souhaitée. Dans le cas contraire, la densité du réseau peut être augmentée sur cette base, en tenant compte des diverses contraintes logistiques, humaines et financières inhérentes au maintien d'un réseau. Mais il y a plus. Les propriétés de l'interpolation optimale permettent, avant même l'implantation de nouvelles stations en des lieux précis, d'analyser l'effet de leur implantation sur le patron d'iso-écart types d'une région. Il suffit, pour celà, de reprendre le calcul des écarts types aux points de grille, en ajoutant aux n stations déjà existantes les m stations projetées, avec leurs positions. On peut aussi, en procédant à une sèrie d'essais, trouver les positions optimales de ces stations (Delhomme et Delfiner, 1973). Une procédure semblable permet d'éliminer des stations au besoin, lorsque la densité de stations est amplement suffisante pour respecter les critères de précision souhaités.

4. CONCLUSION

Il est bien évident que tous les réseaux ne doivent pas être nécessairement conçus suivant l'approche proposée dans cet article. Alors que la prise en compte de certaines des options qui ont été retenues dans la stratégie d'intervention et souhaitable, mais non absolument nécessaire pour assurer une planification rationnelle des réseaux, certains principes fondamentaux devraient par contre être respectés. Le plus important de ces principes est une conception dynamique des réseaux, sans laquelle un réseau demeurerait figé dans l'état où l'a amené la "planification" et ne suivrait pas l'évolution des besoins.

L'importance d'observations effectuées avec soin, recueillies à un nombre suffisant de stations, ne saurait, en outre, être passée sous silence. Sans de telles données, il devient difficile, sinon impossible, d'effectuer une planification de réseau vraiment optimale.
Enfin, l'analyse en composantes principales et l'interpolation optimale se sont avérées être des méthodes complémentaires d'analyse de données, permettant d'appliquer la stratégie de planification des réseaux sur une base régionale, avec la souplesse et la précision désirée, compte tenu des données disponibles. Quant aux résultats, ils doivent être interprétés en fonction des méthodes d'estimation utilisées et des caractéristiques statistiques des données entrées dans l'analyse.

REMERCIMENTS

Les auteurs tiennent à noter que les recherches rapportées dans cet article ont été réalisées grâce à un financement octroyé par le Programme Canadien des Bourses de la francophonie ainsi qu'au conseil de recherche en sciences naturelles et génie du Canada (subvention nº CNR-A-B-540, dont le responsable est José Llamas).

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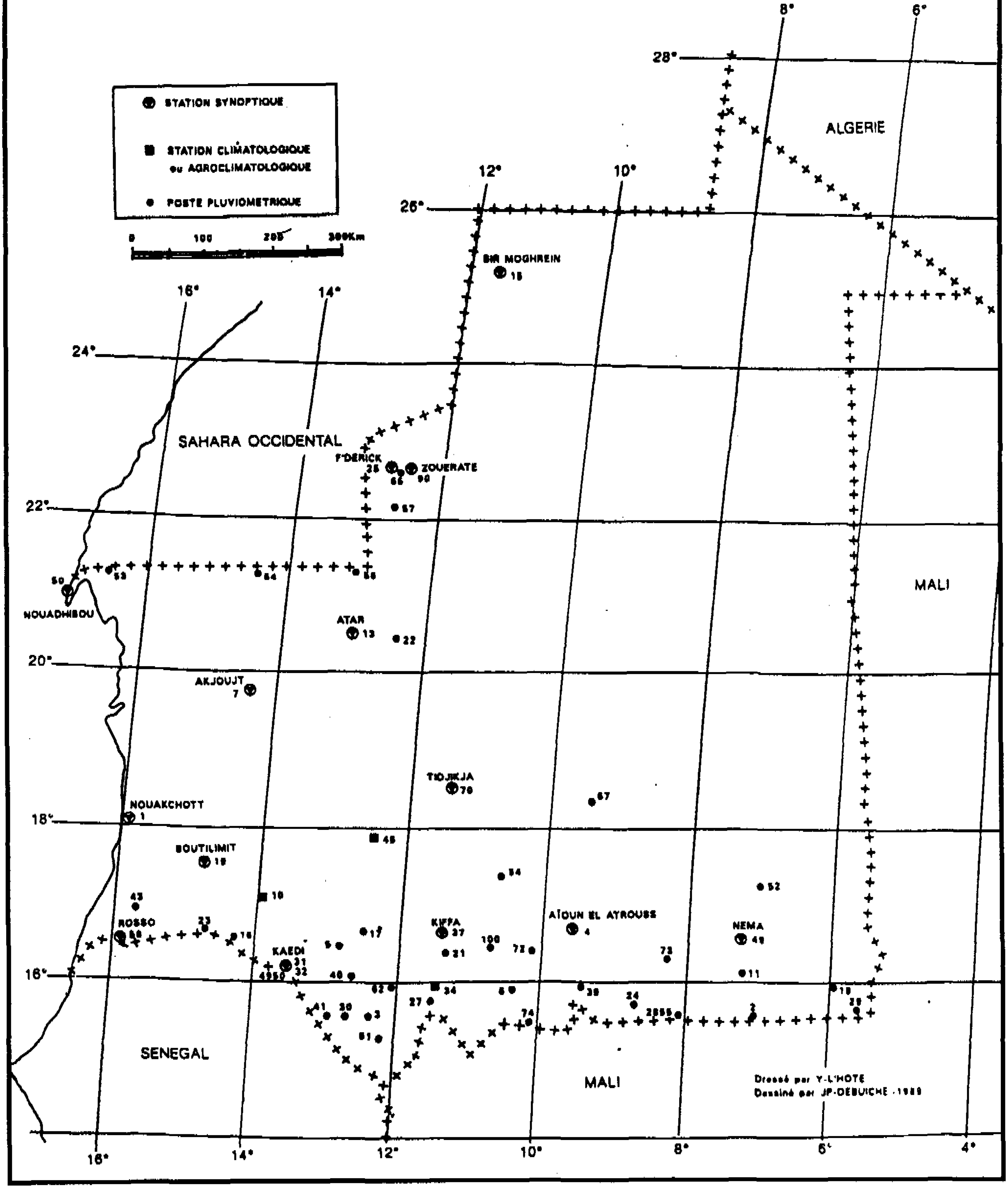
Tableau 1. Corrélation entre composantes et stations "Precipitatins Annuelles"

N° et Nom de la station	Composantes principales						Coeff de correlation multiples
	1	2	3	4	5	6	
1 ALEG	.569	.138	-.137	.479	-.057	.108	.780
2 BOGHÉ	.781	.140	-.162	.354	-.135	-.101	.811
3 TIDJIKJA	.568	.186	-.235	-.165	.527	.116	.779
4 NÉMA	.650	-.407	-.037	-.238	-.087	-.206	.675
5 TIMBEDRA	.545	-.434	-.061	-.283	-.024	.244	.595
6 AIOUNE	.662	-.367	-.339	-.262	.076	-.178	.786
7 KIFFA	.673	-.016	-.282	-.056	-.081	.395	.673
8 KAÉDI	.561	-.189	.464	.192	.241	-.114	.711
9 ROSSO	.710	.088	.111	.318	-.362	-.315	.843
10 BOUTILIMIT	.503	-.415	.591	.108	.068	.290	.844
11 MEDERDRA	.693	.154	-.278	.403	-.037	-.092	.774
12 NOUAKCHOTT	.740	-.024	-.130	-.001	.278	-.289	.720
13 ATAR	.705	.151	.364	-.066	.068	.003	.752
14 NOUADHIBOU	.417	-.029	.342	-.203	-.279	.013	.411
15 SELIBABY	.663	-.255	-.178	.008	-.242	-.023	.704
16 BIR.MOUG	.440	.589	.226	-.226	-.100	.126	.586
17 AKJOUJT	.746	.179	.172	-.096	.138	-.255	.756
18 TAMCHEKATT	.651	-.146	-.101	-.269	-.323	-.137	.666
19 MOUDJER	.559	.206	-.394	.195	-.077	.482	.670
20 CHINGUITI	.610	.461	.132	-.309	-.123	-.060	.737
21 ZOUERATE	.451	.658	.381	-.162	-.012	.166	.597
22 M'BOUT	.567	-.086	.184	.198	.509	-.023	.643
23 TICHITT	.402	-.571	.378	.125	-.109	.282	.756
24 OUALATA	.585	-.021	-.461	-.236	.133	.084	.552
Valeurs Propres	9.00	3.20	2.50	1.99	1.22	1.01	
Variance expliquée (%)	38	13	10.5	8	5	4.5	
Vari-totale expliquée (%)	38	51	61.5	69.5	74.5	79	

Tableau 2. Corrélation entre Stations et composantes après Rotation "Précipitations Annuelles "

N° et Nom de la station	Composantes Principales					
	1	2	3	4	5	6
1 ALEG	.660	.099	.148	.012	.151	.341
2 BOGHÉ	.762	.226	.098	.272	.197	.233
3 TIDJIKJA	.208	.684	.241	-.061	.045	.394
4 NÉMA	.745	.068	.276	.172	.174	-.025
5 TIMBEDRA	.595	.027	.390	-.066	.107	.325
6 AIOUNE	.789	-.045	.062	.160	.339	.155
7 KIFFA	.641	.200	.156	.239	.108	.392
8 KAÉDI	.595	.180	.108	.280	.411	-.169
9 ROSSO	.778	.319	.203	.308	-.011	-.108
10 BOUTILIMIT	.121	.872	.06	.080	.136	.055
11 MEDERDRA	.745	.115	-.001	.202	.260	.278
12 NOUAKCHOTT	.449	.581	.064	.390	.170	.035
13 ATAR	.206	.381	.563	.250	.306	.036
14 NOUADHIBOU	.301	.340	.424	.075	-.125	-.058
15 SELIBABY	.903	.058	.215	.395	.020	.217
16 BIR.MOUG	.001	-.020	.791	.107	.063	.166
17 AKJOUJT	.341	.503	.154	.445	.340	-.068
18 TAMCHEKATT	.275	.702	.110	.250	-.035	.082
19 MOUDJER	.109	.767	.163	.393	.073	.007
20 CHINGUITI	.278	-.044	.759	.186	.137	.087
21 ZOUERATE	-.140	.139	.585	.110	.136	.138
22 M'BOUT	.660	.090	.390	.245	.055	.061
23 TICHITT	.244	.807	-.078	.082	-.058	.104
24 OUALATA	.447	.529	-.130	.123	.339	.101

Figure 1 RESEAU PLUVIOMETRIQUE DE MAURITANIE (Numéros de code ORSTOM)



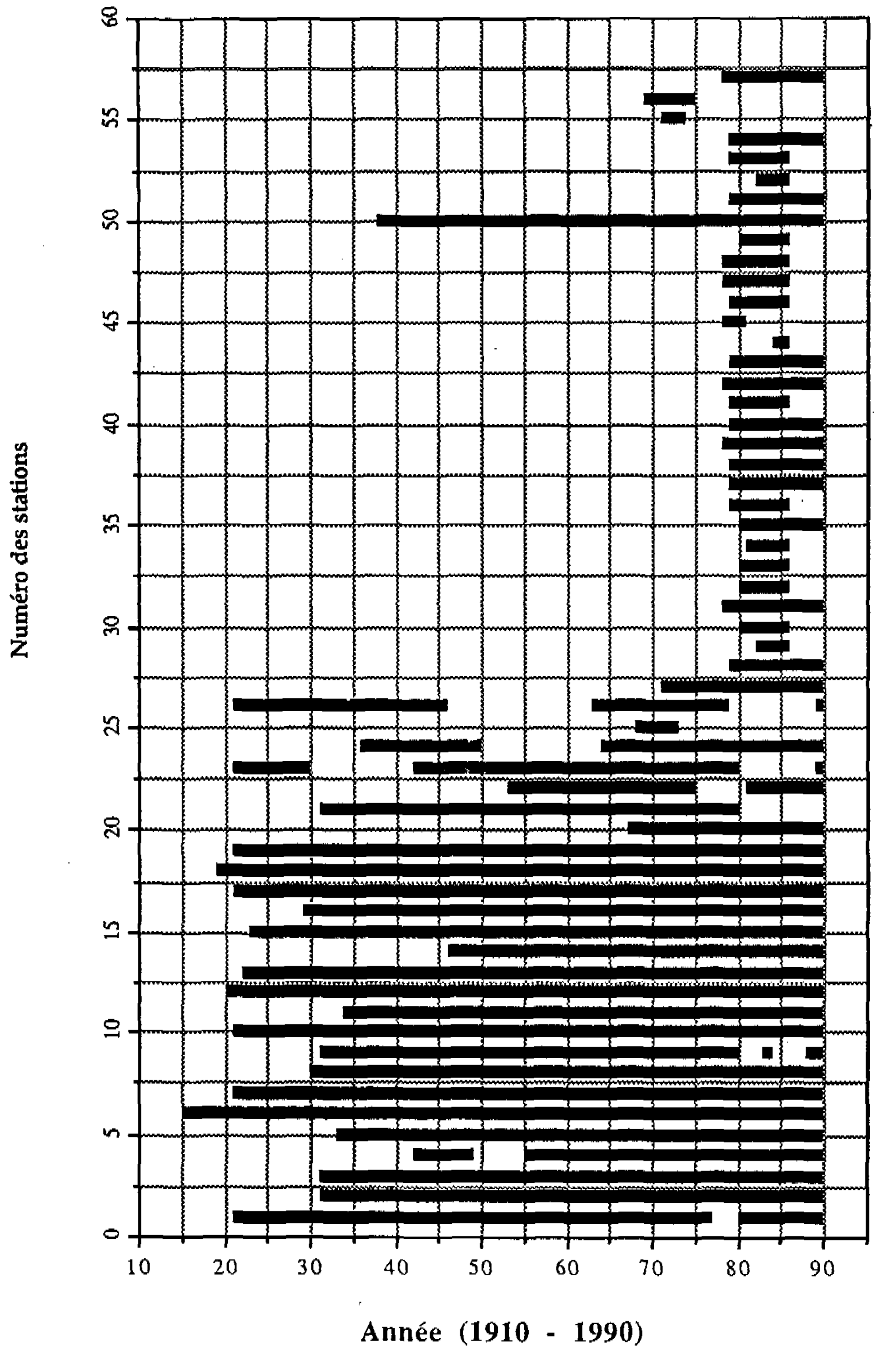
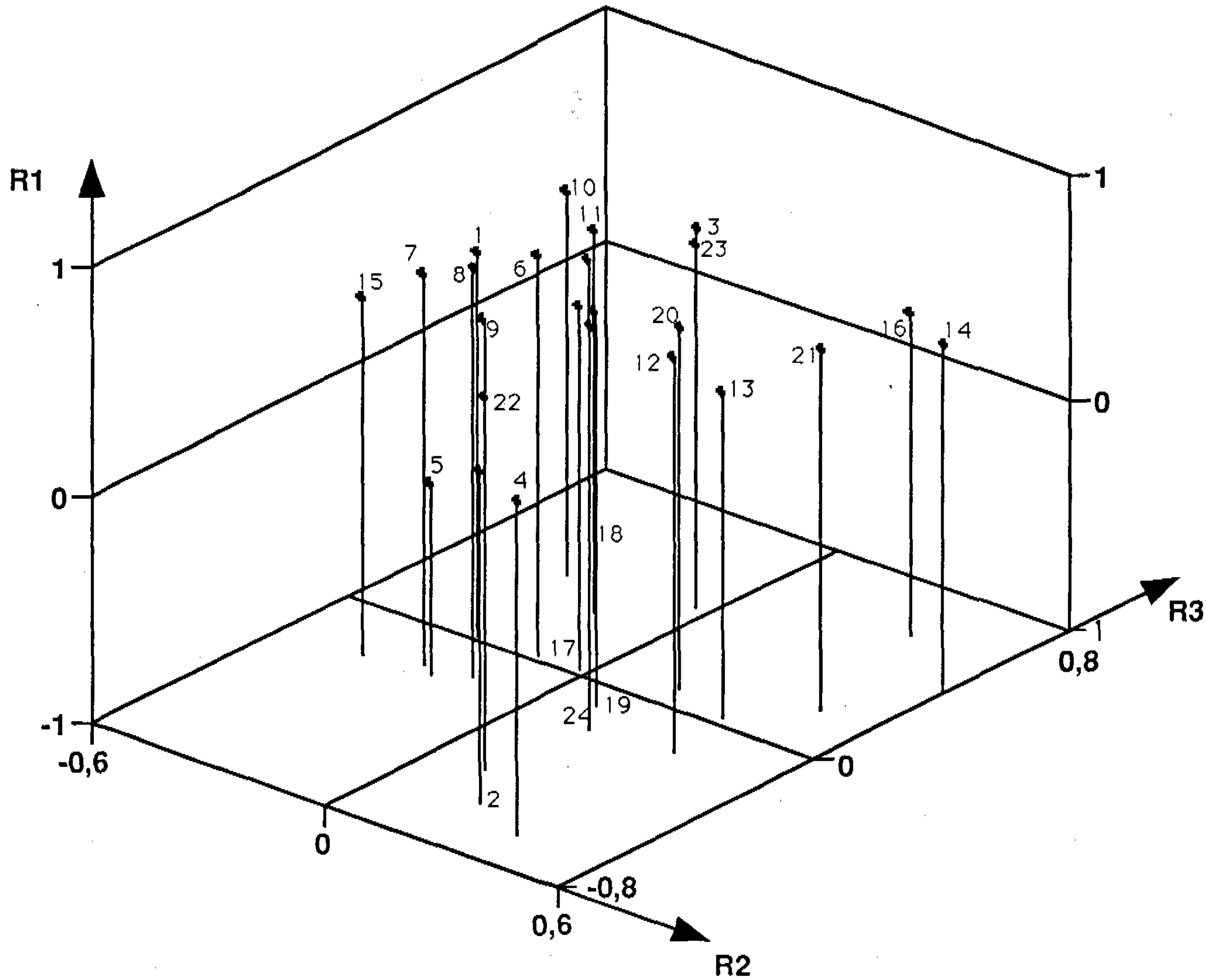


Figure 2. Périodes d'observation des stations



Groupes detectés:

a - 1, 2, 8,9, 22

b - 5, 7, 15

c - 10, 11, 17, 18, 19, 24

d - 3, 12, 13, 20, 23

e - 14, 16, 21

f - 4, 6

Figure 3. Représentation des coefficients de corrélation entre les stations et les trois premiers axes principaux.

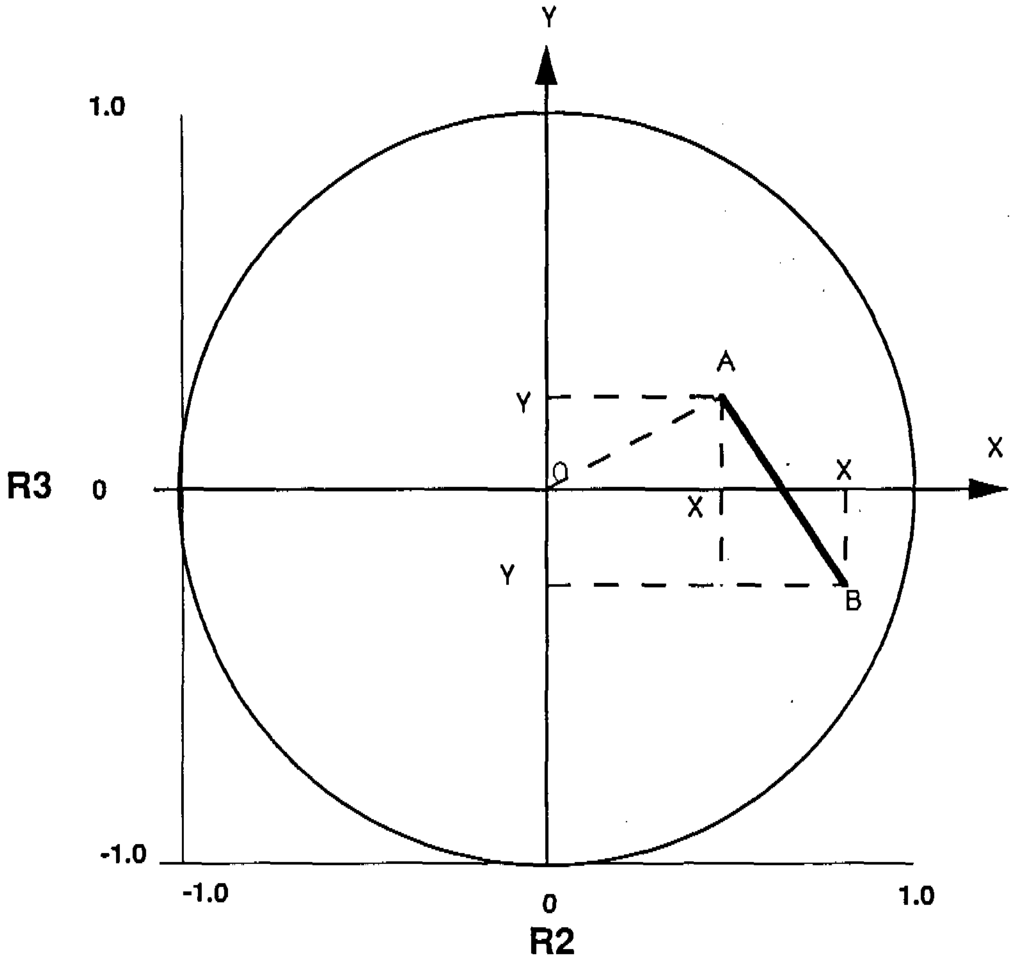
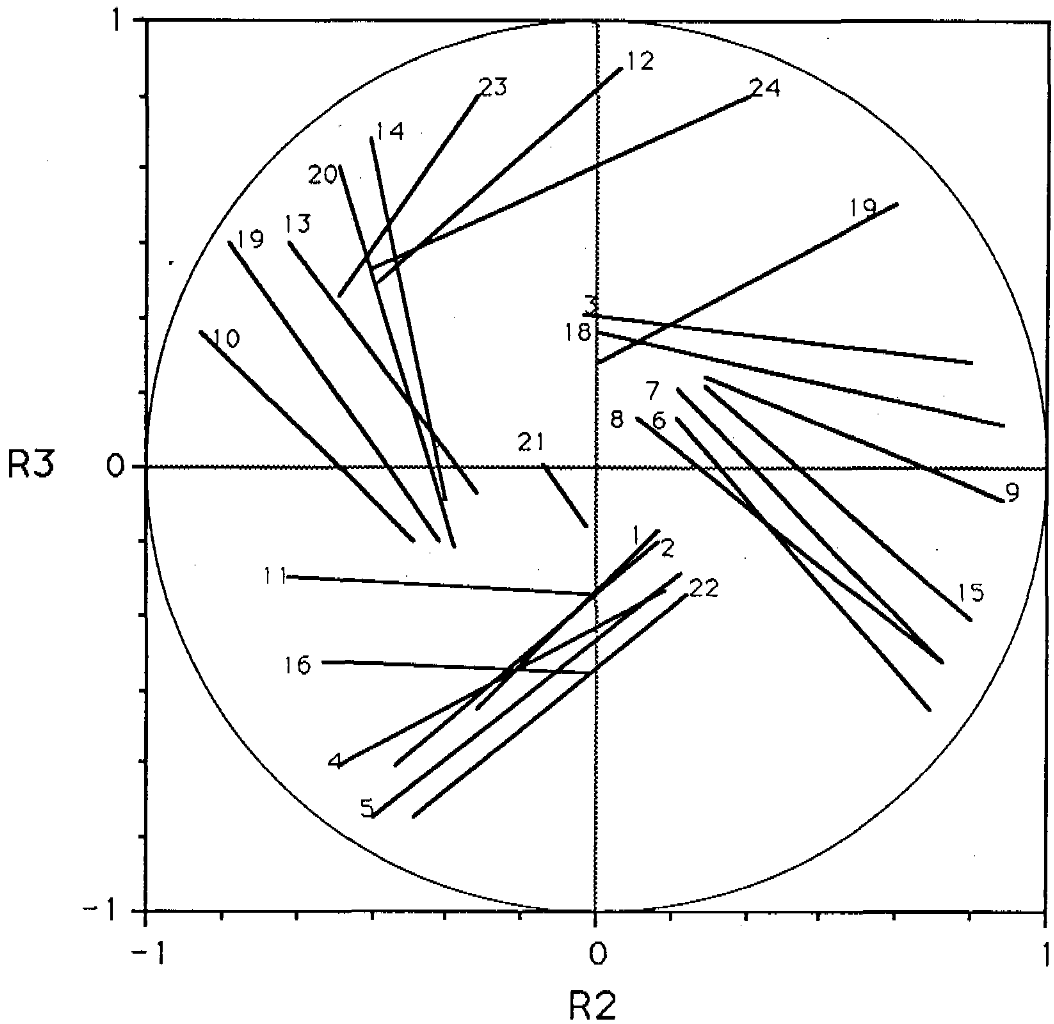


Figure 4. Représentation graphique des coefficients de corrélation entre la station j et les q premières composantes principales, dans le plan des composantes principales 2 (axe X) et 3 (axe Y).



Les groupes:

a - 1, 2, 4, 5, 22

b - 6, 7, 8, 9, 15

c - 3, 18, 19

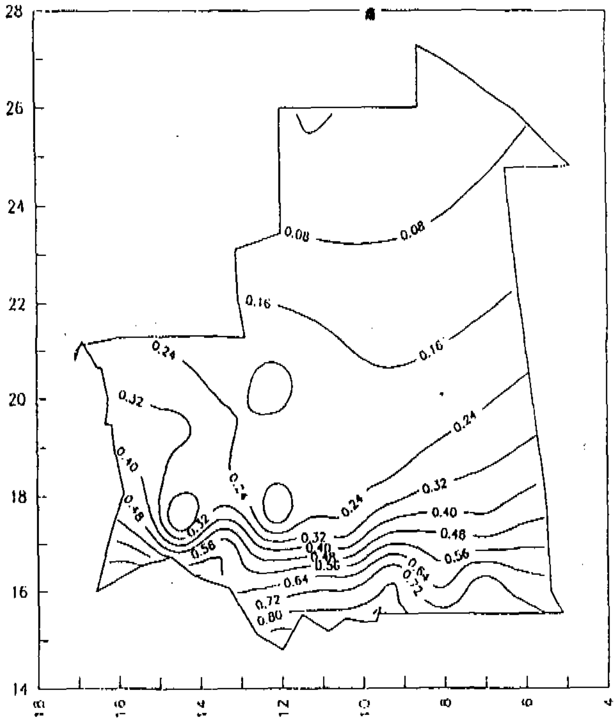
d - 12, 23, 24

e - 13, 14, 20

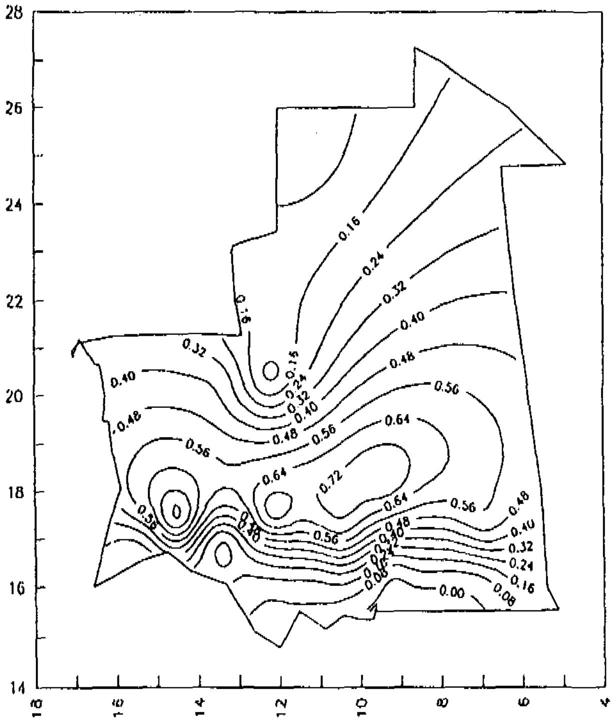
f - 10, 19

Figure 5. Représentation dans le plan des projections des coefficients de corrélation entre les stations et les trois premiers axes principaux. Précipitations annuelles.

Variation spatiale de la première composante principale



Variation spatiale de la deuxième composante principale



Variation spatiale de la troisième composante principale

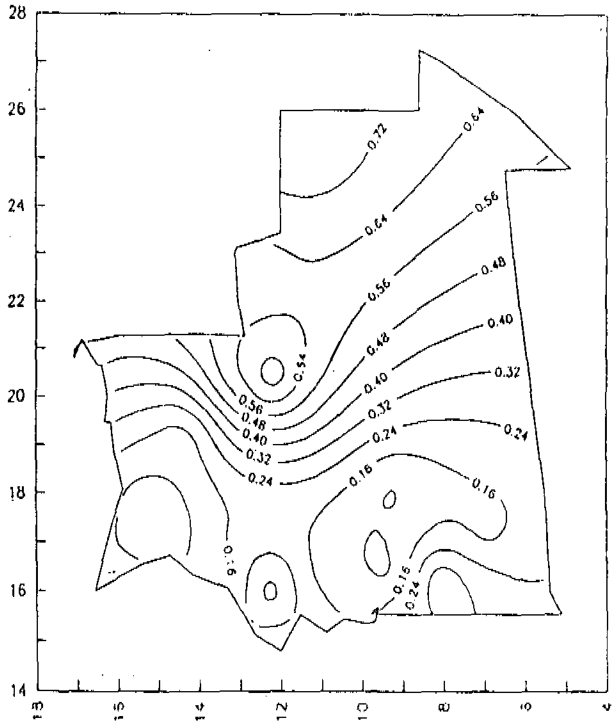


Figure 6. variations spatiales des 3ières composantes principales

(T6-S3) 6.22

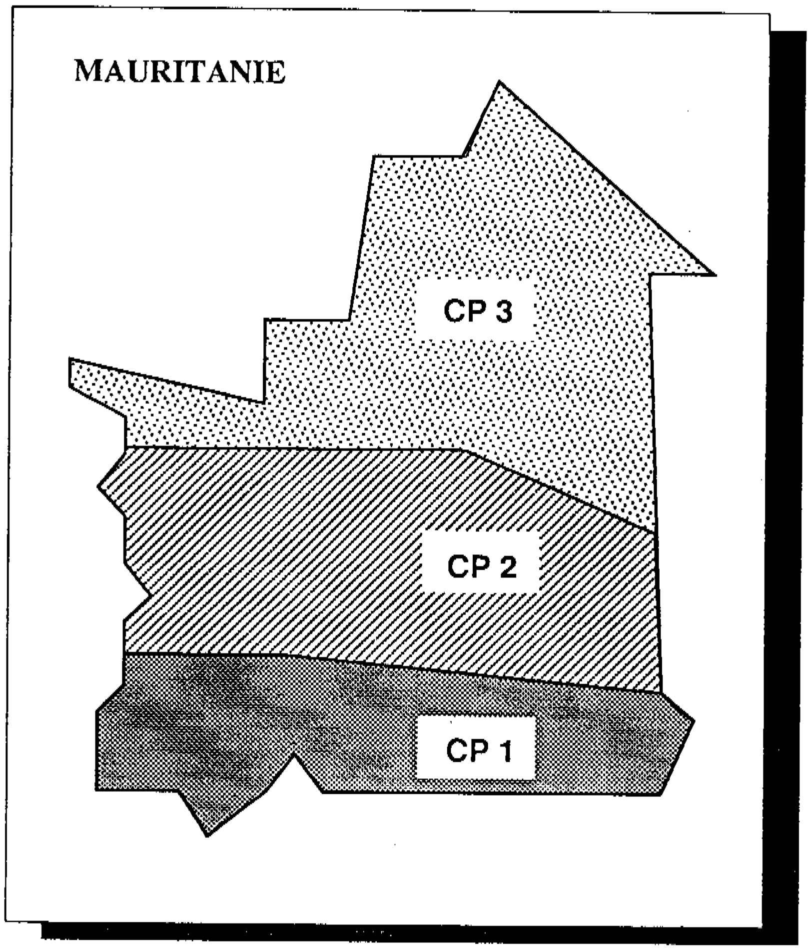
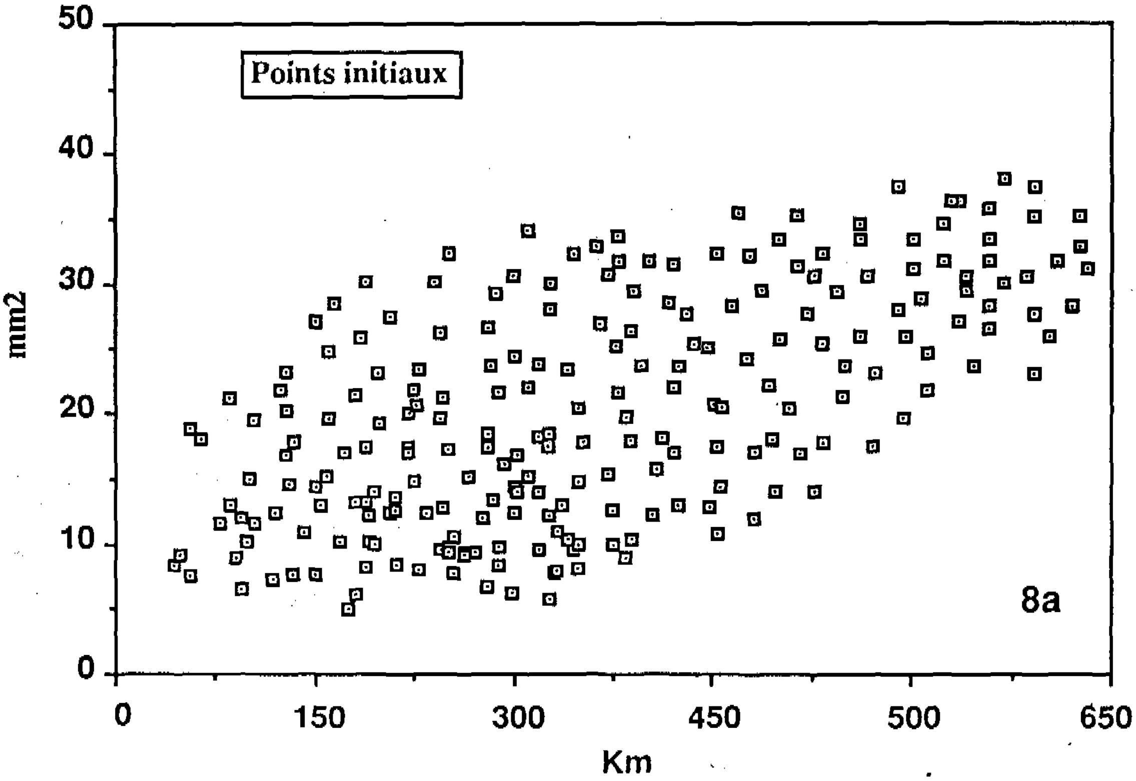


Figure 7. Synthèse des trois premières composantes principales



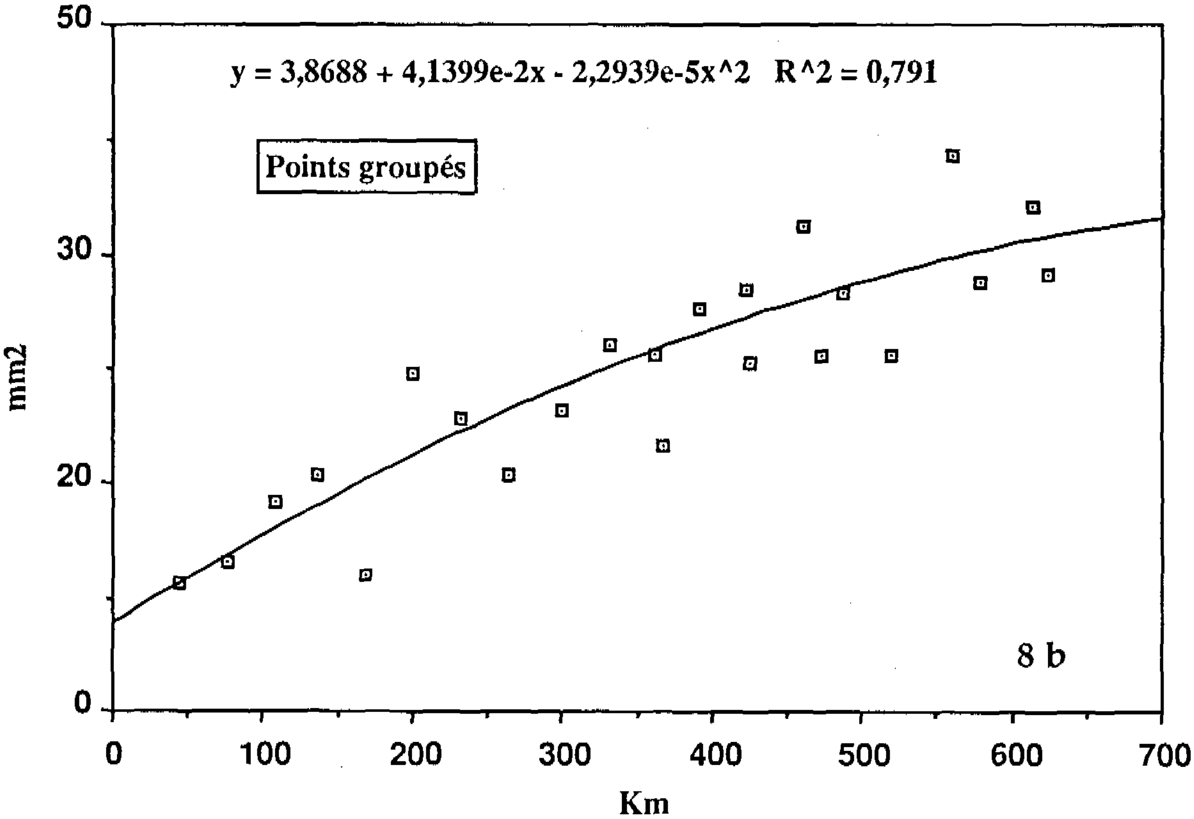


Figure 8. Fonction de structure (mm2) des pluies annuelles, région Sud du pays, en fonction de la distance (km) séparant les stations.

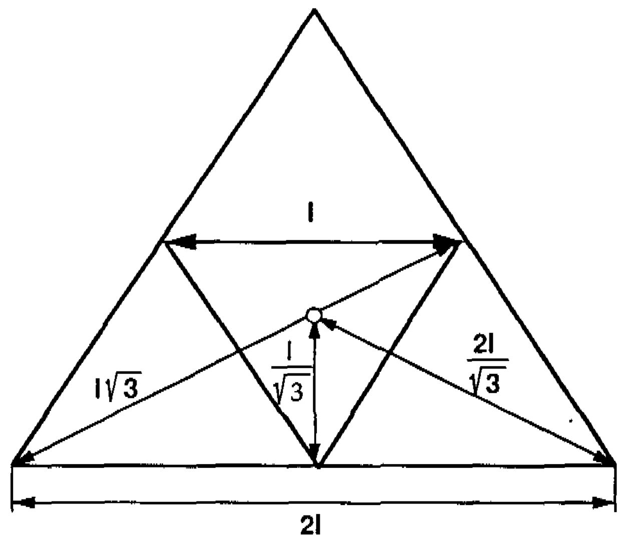


Figure 9. Position des stations pour le calcul de l'erreur d'interpolation au centre de deux triangles équilatéraux emboités.

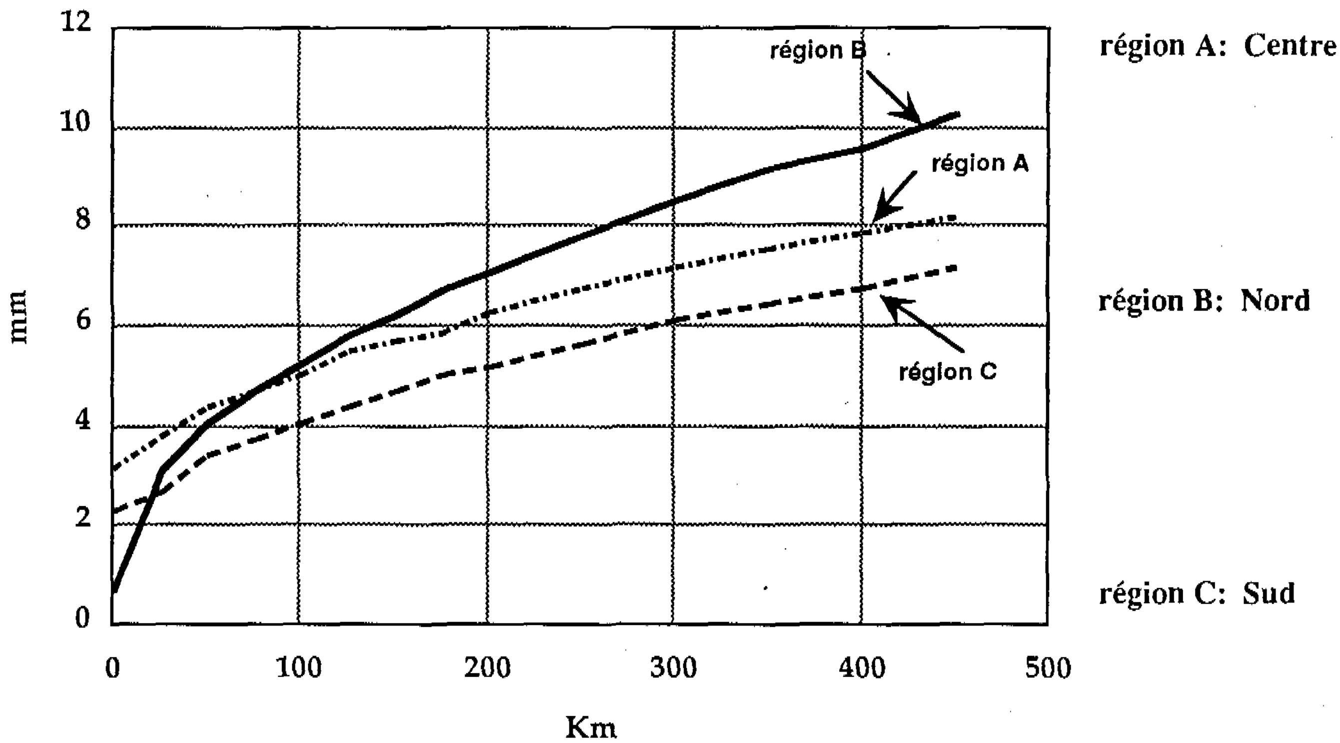
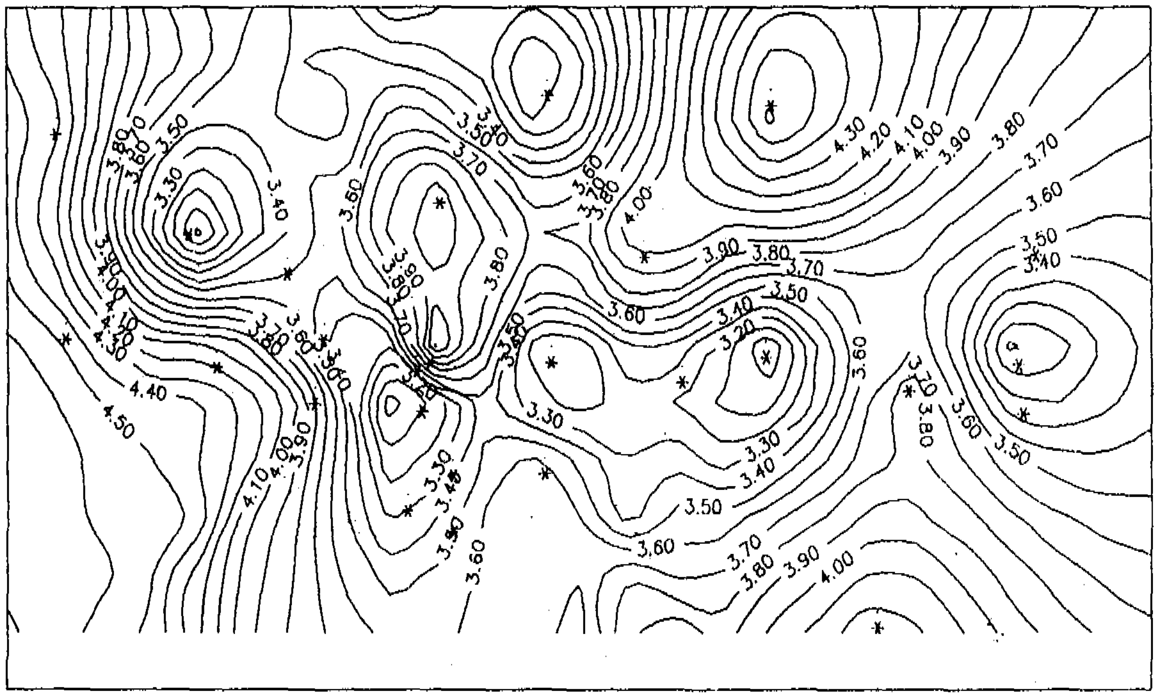


Figure 10. Écart type d'interpolation (mm) en fonction de la distance (Km) pour les pluies annuelles (réseau à mailles triangulaires).

Ecart type d'interpolation pour un reseau de 26 postes



Ecart type d'interpolation pour un reseau de 10 postes

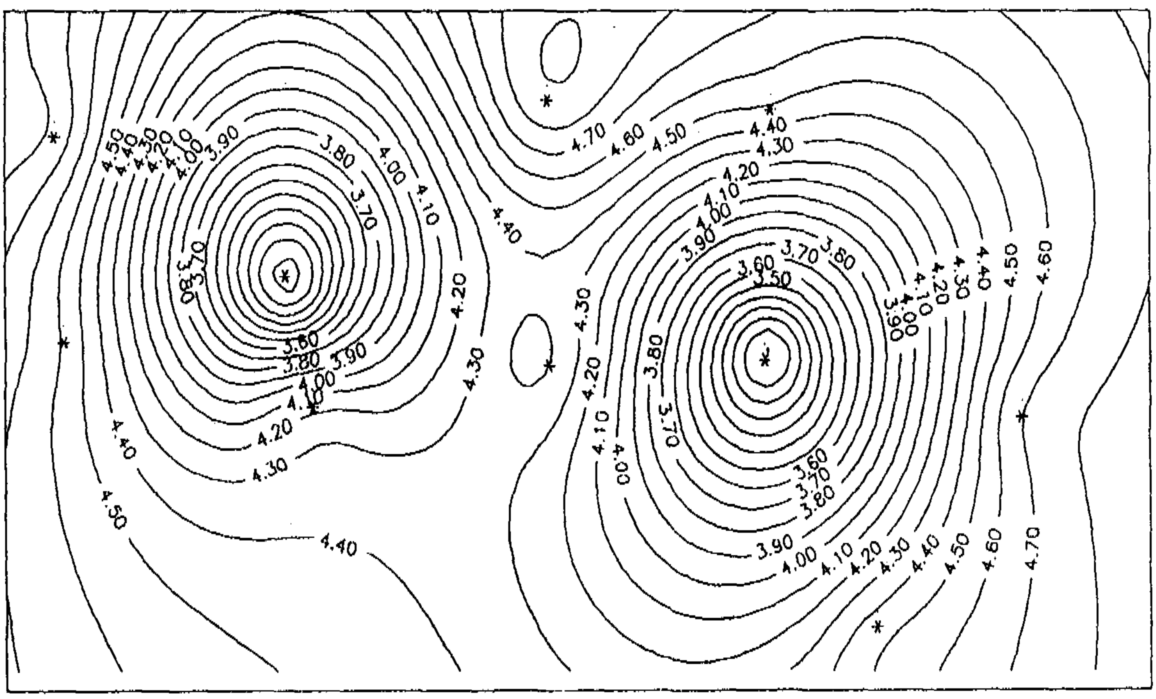


Figure 11. Répartition spatiale des écarts types d'interpolation (mm) des précipitations annuelles pour la région Sud du pays.

SIMULATION OF BED CHANGES AND FLOW CONDITIONS DUE TO SUSPENDED SEDIMENT AT RIVER/RESERVOIR SYSTEM "4-D SIDHOM MODEL"

Mervat S. Sidhom[1], And Gamal S. Ebaid[2]

ABSTRACT

All reservoirs formed by dams on natural waterways are subject to some degree of sediment inflow and deposition.

The accumulation of sediment deposits depends on many interrelated factors such as water and sediment characteristics. so, prediction of the sediment distribution pattern is a complex task because of the interaction between these variables.

The objective of this study is to present quantitative analytical technique for simulating water movement in four dimensions and to evaluate the bed changes of natural waterways as a result of suspended load transport along the stream.

RÉSUMÉ

Tous les réservoirs formés par des digues sur des voies navigables naturelles sont sujets a un afflux d'un certain degré de sédiment et de déposition.

L'accumulation des dépôts de sédiment dépend de plusieurs facteurs interdépendants comme l'eau et les caractéristiques de sédimentation.
Donc, la prédiction d'un modèle de repartition de sédiment est une tâche complexe à cause de l'intéraction entre ces variables.

L'objectif de cette étude est de présenter une technique analitique quantitative de simulation de mouvement d'eau en quatre dimensions et d'évaluer les changements de lits des voies navigables naturelles dû à l'arrêt d'un écoulement chargé le long du fleuve.

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1. INTRODUCTION

Sediment deposition is a troublesome process along natural streams and reservoirs, it raises stream beds, increases flood heights and reduces reservoir capacities and function.

Mathematical models are considered to be recent technological advances in analyzing complicated physical problems.

In order to assess the bed variation in natural streams due to suspended sediment, several mathematical models have been formulated. Below is a list of these models:

- Mathematical model to simulate the water movement in natural water streams and its variation with time. This model is designed to calculate the deformation of the river bed at reservoir inlets.

- Mathematical model for evaluating velocity variation along longitudinal and velocity distribution in vertical and transverse directions in open channels based on the assumption that the velocity distribution in vertical direction is logarithmic. (TRI-SIDHOM MODEL).

- Statistical model for the determination of fall velocity for particles in open channel.

By appling Prandtle-Karman turbulence theory and knowing the concentration of sediment at 0.1 channel depth, and linking the sediment program to the water flow program after damping out the initial conditions for the water flow, then it is easy to compute the value of suspended load by integrating the curve of velocity distribution of water flow and the concentration curve over the depth.

The problem is described by applying the continuity and momentum equations for the water flow, together with continuity equation for suspended sediments.

2. DESCRIPTION OF THE FLOW MODEL AND CALCULATION PROCEDURES

The differential equations of continuity and momentum will be solved in a double tri-diagonal implicit difference scheme and solved with the Double Sweep Algorithm.

The discharges and areas (Q's & A's) are calculated on the network shown in figure (1).

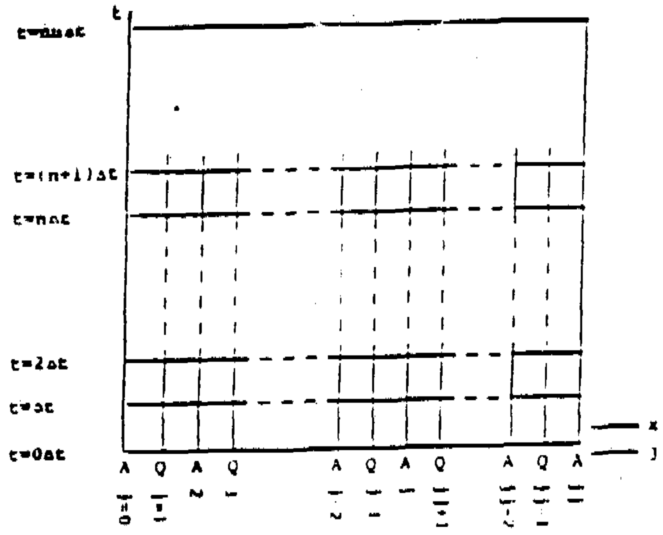


Figure 1. The Network of the Numerical Model.

The points on the line $(n+1)$ are calculated from the points on the line $n\Delta t$, $A^{n+1}{}_{o}$ and $A^{n+1}{}_{jj}$, with the aid of the continuity equation and equation of motion.

Altogether there are $2(jj+1)$ points, unknown are all the points on line $(n+1)$ except $A^{n+1}{}_{o}$ and $A^{n+1}{}_{jj}$ so, there are jj-1 unknowns.

The algorithm will be described as following:
For the use in the Double Sweep Algorithm, the continuity and momentum equation are generalized as:

$$\alpha_j Z^{n+1}{}_{j+1} + \beta_j Z^{n+1}{}_{j} + \gamma_j Z^{n+1}{}_{j-1} = \delta_j \qquad (1)$$

where Z represents Q or A respectively, and α_j,β_j,γ_j and δ_j are the coefficients result from solving the equations system.
Quasi-constants E_j and F_j are introduced in such a way that:

$$Z^{n+1}{}_{j+1} = E_j Z^{n+1}{}_{j} + F_j \qquad (2)$$

Using the boundary condition in point jj, the following values can be set:

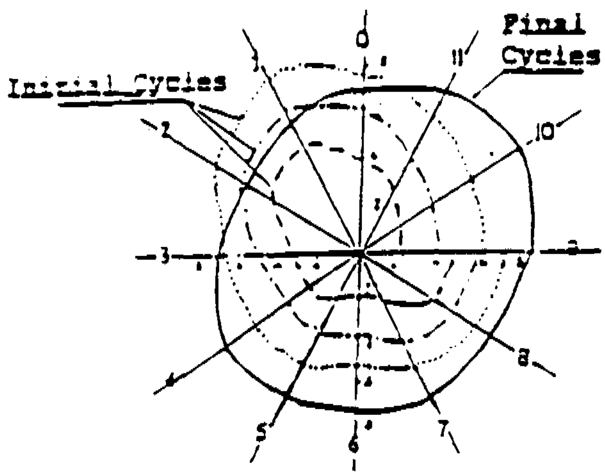
$$E_{jj-1} = 0 \quad \text{and} \quad F_{jj-1} = Z_{jj} \qquad (3)$$

The values of E_j and F_j can be calculated from $j=1$ to $j=jj$.

This cycle is repeated with better values of the coefficients until satisfactory values of Z are obtained. The points on the (n+1) line are then calculated in the following way:

1. From left to right the α, β, γ, and δ constants are calculated from what is known on the (n) and (n+1) line.
2. From right to left the E, and F constants are defined from α_j, β_j, γ_j, δ_j, and A^{n+1}_{jj}.
3. From left to right Q^{n+1}_j and A^{n+1}_j are calculated in turn from A^{n+1}_o and E and F constants.

The results of the flow program gave the possibility to plot the variation of water depths or water velocities at various locations of the channel with time. It is clear that the effect of initial velocity is damped out as in figure (2).



------- u_o = 0.2 m/sec.
- - - u_o = 0.4 m/sec.
........u_o = 0.6 m/sec.

Figure 2. Damping of Initial Velocity,u_o,(Polar Representation)

3. SPACE DIMENSIONAL DISTRIBUTION OF VELOCITY IN OPEN CHANNEL "TRI-SIDHOM MODEL"

Vanoni has demonstrated the vertical velocity distribution law in terms of the mean velocity V_{mean}.

$$V = V_{mean} + (1/K)(g.d.S)^{1/2} (1+2.3 \log_{10}(y/d)) \tag{4}$$

where: d is the depth of the water in the canal, V is the velocity at a distance y from the canal bed, V_{max} is the maximum velocity at the considered vertical section, S is the water slope, and g is the gravitational acceleration.

Vanoni's equation leads to the statistical model(TRI-SIDHOM):

$$V_{ijk} = \mu_i + \beta_i X_{ijk} + \gamma_i h_{ijk} + e_{ijk} \tag{5}$$

where: V_{ijk} is the measured velocity at the k-th relative depth at the j-th relative horizontal distance at the i-th cross section of the reservoir, μ_i is the mean velocity at the i-th cross section, β_i, is 7.83 times the square root of the slope at the i-th cross section, γ_i is the regression coefficient at the i-th cross section, $X_{ijk} = (D_{ij})^{1/2} [1+(2.3)\log(1-R_{ijk})]$, h_{ijk} is the relative horizontal distance to closest bank at the j-th position, at i-th cross section, and e_{ijk} is random error.

The distribution of velocity in horizontal can be estimated from the fitted equation:

$$V_{ijk} = \mu_i + \beta_i X_{ijk} + \gamma_i h_{ijk} \tag{6}$$

4. PROCEDURE OF CALCULATION (METHODOLOGY)

To determine the horizontal velocity distribution at any cross section at any vertical depth D, at any horizontal distance h, one can follow the following procedure:
Fix a cross section i:

1. Pick a fixed relative depth R_o.
2. Consider all relative horizontal distance h, $0 < h \leq 0.5$.
3. For each h, let D(h) be the total depth h (read map).
4. Let $X = (D(h))^{1/2}[1+(2.3)\log(1-R)]$

5. Plot V(h) versus h, where μ_i, β_i, γ_i are coefficients that result from the High Aswan Dam Reservoir's cross sections data This procedure of the calculations is described in figure (3).

By linking the modified Vanoni's equation (TRI-SIDHOM Model) to the main Double Sweep Algorithm, one can get the full description of flow in four dimensions at any required section, (vertical, lateral, and/or horizontal) at any time increment.

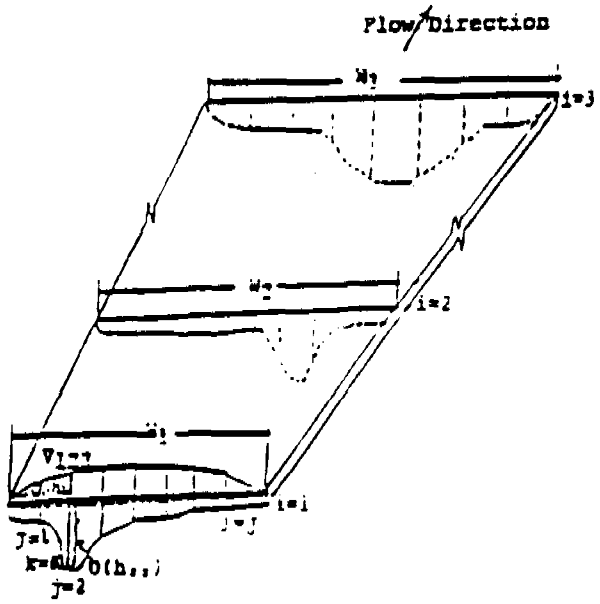


Figure 3. Illustration of Horizontal Velocity Distribution at Cross Section of a Reservoir.

5. SUSPENDED SEDIMENT TRANSPORT EQUATIONS IN NATURAL WATER STREAMS AND THEIR MATHEMATICAL SIMULATION

The following formula represents the physical law controlling the distribution of the suspended sediment over the vertical:

$$n/n_a = [((y_m-y)/y).(a/(y_m-a))]^{w/(k(\tau o/\rho)^{1/2})} = N^z \qquad (7)$$

in which:

n_a is the concentration at an arbitrary height, a.
k is Von karman's universal constant (0.4 for pipes)
U. is the sear velocity as it is equal to $(\tau_0/\rho)^{1/2}$
y_m is the value of y for which the velocity is maximum
w is the fall velocity of the grain.

Thus, to solve the problem of the calculation of suspended sediment distribution over the vertical, we must know the grain size distribution curve of the area under consideration to find the representative diameter which is considered as d_{50}, Also, we have to know the relation of settling velocity and the grain diameter, which found to be a polynomial equation of fourth degree:

$$\log x = a_o + a_1(\log y) + a_2(\log y)^2 + a_3(\log y)^3 + a_4(\log y)^4 \qquad (8)$$

in which y represents the value of particle diameter in millimeters, x is the corresponding value of fall velocity in centimeters/sec., a_o, a_1, a_2, a_3, and a_4 are the coefficients of the fitted equation.

Thus one know the value of the coefficient "ξ" which equal to $w/k(\tau_o/\rho)^{1/2}$ .Thus, the value of total suspended load could be computed by integrating the product of velocity and concentration over the total depth then getting:

$$T_s = \sum_{\text{total}}^{\text{depth}} n.\ V.\ \Delta y \qquad (9)$$

where: n is the concentration of suspended sediment at depth y,
and V is the value of velocity at depth y.

Therefore, the main program can describe the velocity distribution in three dimensions, concentration curve of suspended load and the total amount of material transported.

The variation in bed levels resulting from changes in suspended load transport at various sections was determined from applying the continuity equation of sediments.

The change in bed level between any two successive sections is equal to the difference between mass transported in and out of these sections.

$$[(\sum_{\text{bed}}^{\text{surface}} V.\ n'.\ B_1.\ \Delta y)_{sec1} - (\sum_{\text{bed}}^{\text{surface}} V.\ n'.\ B_2.\ \Delta y)_{sec2}].\Delta t = DZ.(B^1+B_2)/2).L \qquad (10)$$

where DZ is the variation in bed levels
B_1 & B_2 are the cross section widths
L is the length of the reach under consideration.

The result of this study to the given reach along the River Nile under the harmonic water level changes at inlet section, is shown in figure (4). This figure represents the variation of "DZ" along the channel length. Thus the variation in the bed level is then calculated and consequently the new depths then calculated by adding the value of "DZ" to the previous depth in the new cycle of the computation.

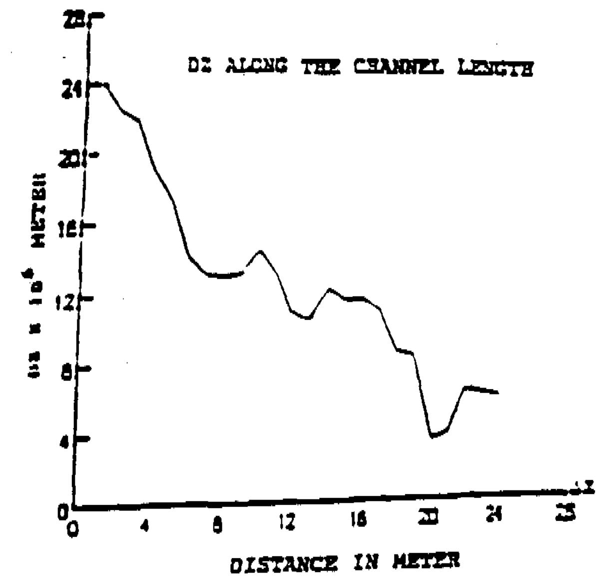


Figure 4. Variation of DZ Along the Channel Length.

6. ABRUPT CHANGES OF THE RIVER CHANNEL SECTION

Some times, for certain parts of the river streams, up to several Kilometers, an abrupt changes in the cross section, takes place, this means hat the increase of the water section with respect to the original one is considerably big.

Accordingly, the existence of such sudden changes of cross sections are considered a silt trap, in which the whole sediment are found to be accumulated, and the flowing water to the next reaches will be almost free of sediments.

As a result of this trapping of sediment, the cross section in this particular place will be decreased, and the flow will enter a new stage of behavior, that causes either degradation, i.e. carrying out the new sediment material to the next reaches, or create some swamps in these places. In either cases, training works for the river streams, in such areas, is recommended in order to keep the continuity of sediment movement and facilitate their right track of deposition. To get rid of this problem, the training works must be designed across the stream in such areas, to define a certain route for the flow.

7. CONCLUSIONS

1. It is clear that mathematical models could be successfully used for the study of unsteady flow problems.

2. The implicit difference scheme used for the differential equations in the mathematical model is unconditionally stable and save too much computer time.

3. Vanoni's equation can be modified to determine not only the vertical velocity distribution, but also the horizontal (longitudinal and lateral) velocity distribution in open channels (TRI-SIDHOM Model).

4. TRI-SIDHOM Model can be used to determine the velocity distribution in four dimensions when we introduce time factor as the fourth dimension.

5. It is easy to compute the value of suspended load by integrating the velocity distribution curve and the concentration curve over the depth. Thus the variation in bed levels can be calculated.

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LOW SALINE GROUNDWATER IN EGYPT AND SATISFYING DEMANDS IN THE NEW AGRICULTURAL LAND AREAS

Abdu A. Shata*

ABSTRACT

Low saline groundwater, having reserve estimates in excess of 500 milliard cubic meters is little considered in the reclamation of the new land areas. This type of water has a wide geographical distribution in Egypt and exists, generally, at shallow exploitable depths. Main areas of interest are the coastal desert regions, the old alluvial plains of the Nile and Delta and the fissured porous sandstones and carbonates in the inland desert regions. Assuming satisfying demand rates of this type of water in non-traditional agriculture amounting to 5000 cubic meters per feddan per year, Egypt can add about one million feddans to its green area. Detailed hydrogeological studies are wanted to make better assessement and better evaluation of this water for sustainable development.

BASSES EAUX SALLEES ET NOUVELLES METHODES POUR UNE MEILLEUR AGRICULTURE DES TERRAINS EGYPTIENS

RÉSUMÉ

Bien que notre reserve d'eau souterrain depasse 500 milliard cubic meters ,son exploitation dans la reclamation des nouveaux terrains, reste fortement modeste. Cette eau souterraine, repondue geographical ement sur notre terrain, se trouve en general a des niveaux peu profonds facilement exploites. Les centres d'interet sont les regions desertiques cotiers, les anciennes plaines alluviales du Nil, de meme que les carbonates fissures dans les regions fluviales. Les basses eaux, emplement suffisant a l'agricultures non-tradionelle 5000 cubic meters par feddan par an, l'Egypte pouvait, bien ajouter 1 million de feddans a ses surfaces vertes. Les etudes hydrogeologiques detaillees sont donc indispensables pour definir un plan de development a long terme.

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1. INTRODUCTION

By the year 2000, the population of Egypt will be in excess of 75 million and the volume of fresh water that can be made available, essentially for traditional agricultural development, will remain almost unchanged. This comprises:-

- 55 millard cubic meters of surface water per year, as laid down in the Nile Water Agreement with Sudan,
- 3 milliard cubic meters of groundwater per year (can be increased to 5 and 7 milliard) both from the geological formations underneath the Nile and Delta (renewable water) and undermeath the inland desert areas west of the Nile (artesian and mainly non-renewable),
- 0.3 milliard cubic meters of surface runoff water from local rain

In view of the continued losses of surface water (mainly during conveyance and which amouts to about 10 milliard m3 per year), the questionable conservation schemes of the Jonglei Canal and the inadequate use of groundwater, the future of traditional agricultural development in Egypt is rather limited. This may mean that the use of the vast amounts of low saline groundwater in non-traditional agricultural development will become inevitable in the coming years.

From the regional geological and hydrogeological mapping activities of the past decades, we are fairly well informed about the geographical distribution of low saline water in Egypt, but so far no real assessements have been attempted. In an approach to improve knowledge about the potentiality of this type of water and its use in the Nile Valley and the Delta,as well as in the desert areas beyond it, a discussion of the following topics shall be made :

- the regional hydrogeology,
- the regional groundwater potentiality,
- the occurrences of low saline groundwater, and
- the irrigation demand possibilities.

2. REGIONAL HYDROGEOLOGY

In reference to the regional hydrogeological map of Egypt, scale 1:2,000,000 (RIGW, 1988), the water-bearing strata are as follows, arranged in a descending chronological order :

2.1 The Quaternary-Upper Tertiary fluviatile sands and gravels in the Nile and Delta Basins; having a saturated thickness varying from less than 100 m to more than 500 m. These form a common hydrogeological unit, which is extensive and is highly productive. Porosity measurements of this unit vary from 20 % to 40 % , the transmissivity is in the order of $\pm$ 5000 m2/d and the conductiviy is about 60 m/d . This unit is almost saturated with both fresh water on top and low saline water at the bottom. The water is both confined and non-confined and the direction of flow is regionally in the northward direction, i.e. to the Mediterranean Sea, which acts as a main discharging basin. .

2.2 The Quaternary - Upper Tertiary shallow marine calcareous sandstone and fluvio-marine sandstone and gravel in the coastal desert areas of both the Mediteranean and the Red Sea regions; having a saturated thickness, normally less than 100 m. These strata form a common hydrogeological unit, which is extensive and is moderately productive. Porosity measurements are about 20 percent and the transmissivity is generally less than 500 m2/d. This unit is saturated with both brackish water on top and with saline water below. The water is mainly renewable. The water is generally unconfined and the flow pattern is to the north, i.e. in the direction of the Mediterranean Basin, to the east and also to the west in the direction of the rifted Red Sea Basin.

2.3 The Miocene-Oligocene fluviomarine, sands and gravels, in the region west of the Nile Delta and its westward extention to the great Qattara Depression; having a saturated thickness in excess of 200 m. These form a regional hydrogeological unit, which occupies wide tracts in the northern portion of the Western Desert and which represants the delta of an ancient river, during the Oligocene and the early Miocene. Porosity measurements are in the order of 20 per cent and the transmissivity is $\pm$ 500 m2/d. This unit is extensive and is moderately productive. The occurring groundwater is mainly brackish and saline and exists generally under semi-confined conditions. The flow pattern is in the westward direction, i.e. to the great Qattara Depression, which acts as a discharging area.

2.4 The Eocene-Upper Cretaceous fissured carbonate rocks; having a thickness in excess of 1000m. These form a major multi-layered hydrogeological unit, which occupies about 50 % of the total area of Egypt. It displays classical examples of irregular paleo-karstified features and there are records of springs located at different altitudes and tapping low saline to highly saline water. This unit is least studied in Egypt and there is accordingly lack of hydrological information.

2.5 The Mesozoic-Paleozoic sandstones (undifferentiated); having a thickness varying from less than 500 m to more than 3000 m. These form one of the most extensive hydrogeological units in Egypt, and its productivity varies from high to moderate. This unit occupies more than 50 perecent of the total area of Egypt and contains generally fresh water in the central and the southwestern portions, brackish water in the eastern portions and saline to hypersaline water in the northern prortions. This sandstone forms a regional hydrogeological basin, which extends outside Egypt into the Sudan and Libya. In this basin the section is locally differentiated into different levels, which are regionally in hydraulic connection. Both primary and secondary porosity features are recorded and vary from less than 10 % to more than 30 % . The transmissivity values are variable and are in excess of 5000 m2/d. The water is generally nonrenewable, exists regionaly under artesian conditions and the flow pattern is in the northeast direction (local complexities are wittnessed). The Qattara depression is described as a major discharging area for this basin.

2.6 The Precambrian fissured Basement (hard rocks); occupying about 10 % of the area of Egypt. These form another hydrogeological unit, but it is also least studied in Egypt. Local occurrences of groundwater are found in it and the productivity is rather limited.

3- GROUNDWATER POTENTIALITY

Considering the regional geographical distribution of the main hydrogeological units, rough estimates of the groundwater reserves are made by some workers including Shata (1982) and RIGW (1992). Such reserves are in excess of 50,000 milliard cubic meters. The potentiality of this geat amount of water, which is both renewable and non-renewable is, classically, determined in view of the quality, the cost of pumping, the depletion of storage and eventnally the economic return over a fixed period of time. Arranged in a descending order, the potentiaility of the main aquifer systems in Egypt is as follows (Table 1):-

3-1 The Paleozoic - Mesozoic Sandstone Aquifer System in the inland desert areas, having reserve estimates of more than 40,000 milliard m3, in an area of about 500,000 km2. The water is mainly artesian and is essentially non-renewable. Maximum potentiality is restricted to the natural depressions in the Western Desert, which are at or below sea level and where the water is still flowing to the surface or can be tapped at shallow depths below the surface (generally less than 100m) . The present extraction is of the order of 700 million m3 / year (about 2.0 million m3/ day). This amount can be increased to 5000 million m3 / year in the coming century.

3-2 The Quaternary Alluvial Aquifer System in the Nile-Delta Basin; having reserve estimates of about 500 milliard m3 in an area of 50,000 Km2. This water is essentially renewable and the recharge is from the Nile. This water is practically available in the whole area and the pumping depths are much less than 100m. The present extraction is of the order of 3.0 milliard m3 / year (10.0 million m3/day). This amount is expected to increase, progressively, to 5.0 milliard m3 and to 7.0 milliard m3 / y during the coming century .

3.3 The Quaternary - Late Tertiary Clastic Aquifer Systems in the coastal desert areas; having reserve estimates of about 200 milliard m3 in an area of about 40,000 Km2. This water is almost renewable and the recharge is from the local rainfall and the surface runoff water. This water is practically available in the whole area and the pumping depths are less than 100m. The present extraction is about 100 million m3 / year (300,000 m3/day). This amount is expected to increase to 100 million m3 / year in the coming century .

3-4 The Miocene - Oligocene Fluviomarine Aquifer System in the region west of the Nile Delta (Moghra Aquifer System); having reserve estimates of about 100 milliard m3 in an area of about 20,000 Km2. This water is almost nonrenewable and the pumping depths vary from less than 100m to more than 200m. The present extraction is about 150 million m3 / year (0.5 million m3/ day). This amount is expected to increase to 300 million m3/year in the coming century.

TABLE ONE

GROUNDWATER RESOURCES
OF
EGYPT

AQUIFER SYSTEMS	STORAGE CAPACITY (MILLIARD M^3)	WATER QUALITY	PRESENT EXTRACTION		FUTURE EXTRACTION (MILLION M^3/Y)
			MILLION M^3/D	MILLION M^3/Y	
Nubian Sandstone	> 50,000	50% F	2.0	700.0	3000 to 5000
Nile And Delta	>500.0	50% F	10.0	3000.0	5000 to 7000
Coastal	>200.0	100% B+S	0.35	120.0	200.0
Ancient Delta (Moghra)	>100.0	50% B+S	0.50	175.0	300.0
Fissured Carbonates	>100.0	100% B+S	0.50	175.0	300.0
Fissured Hard Rocks	> 10.0	100% F	0.03	10.0	20.0
Total	> 51,000	-	13.38	4180.0	8820 to 12820

NOTES :

1. F = Fresh (ppm < 1000), B = Brackish (ppm < 5000) and S = Saline (ppm > 5000).
2. Estimated volume of brackish water in storage is of the order of 5000 million m^3.
3. Estimated volume of brackish water available for use is of the order of 2000 million m^3.

3-5 The Eocene-Upper Cretaceous Fissured Carbonate Aquifer System; occupying an area of about 500,000 Km2 and is dominantly exposed in the extensive flat-topped plateaux in the area west of the Nile, between the Nile and the Red Sea Hills and in Sinai. Little information is available about the hydrogeology of the carbonate rocks. The rain fall is moderate in north Egypt (50 - 100mm) and is limited in the south. Bare limestone surfaces, have very limited retention (estimeted as 5mm in Libya). Thus it is expected that most of the storm water (in excess of 10 mm /day) is likely to infilterate into the fissured system prior to its drainage by the diffent wadis. Information obtained from scattered springs and from wells tapping the fissured carbonates in Sinai and the Western Desert to the west of the Qattara Depression, shows that there are potential occurrances of groundwater at about sea level. Assuming a groundwater flow of about 2mm, this would mean a recharge of about 500 million m3/year in north Egypt (north of lat. 28° N). The exploitation potential of this type of water is uncertain and is probably very irregular due to the degree of fracturation. The reserve estimates are of the order of 500 to 1000 milliard m3 and the daily out flow is roughly 1.0 million m3 (300 million m3/y). This amount may remain constant or can be increased to 500 million m3/year in the coming century.

3-6 The Fissured Hard Rock Aquifer System (Basement Rocks); having limited potentiality and further hydrogeological exploratory work is wanted. Local occurrences are restricted to South Sinai and to the Red Sea Hills in the Eastern Desert. The present extraction is only of the order of 30 million m3/y, and is esseatially used in the mining areas and for the municipality (100,000 m3/day). This amount can be increased to 50.0 million m3/year in the coming century .

4 - LOW SALINE GROUNDWATER OCCURRENCES

In Egypt the reserve estimates of both the renewable and the nonrenewable groundwater are in excess of 50,000 milliard m3, of which only a small proportion is in use or flows naturally to the surface. This proportion, amounting to less than 4.0 milliard m3/year, is equivalent to 0.002 per cent of the reserves and comprises essentially the good quality groundwater. Although no efforts are directed to make an assessement and to make prediction of the future possibilities for using the low saline groundwater in Egypt, reference can be made, at this early stage of our knowledge to the following areas of interest:

- the coastal desert areas ,
- the fringes of the Nile Valley and Delta,
- the eastern portion of an ancient delta located to the south and east of the Qattara Depession,
- the north and eastern periferies of the great Nubian Sandstone Basin,and
- the lowland areas to the south and west of the geat Qattara Depression.

4.1 The Coastal Desert Areas

In the coastal desert areas of Egypt, there is complete lack of fresh groundwater. On the other hand there are vast amounts of low saline groundwater with salinities, varying between 1000 ppm and less than 5000 ppm. The renewable reserves are about 200 milliard m3 and the daily extraction is very limited and amounts to 300,000 m3. The water exists under both phreatic and semiconfined conditions. In such areas there is a general hydrodynamic equilibrium with the saline groundwater below. The main geographical and geological occurrences of this type of water comprise the following (Figure 1) :-

* The Coastal Calcareous Ridges; these dominate the area to the west of the Nile Delta and are locally found far to the east of the Nile Delta in Sinai. The water is obtained from excavated galleries (length about 16 km) and from both drilled and hand dug wells (> 3000 wells). The daily extraction is of the order of 100,000 m3 and the estimated long term production can be increased to 250,000 m3/day (85 million m3/year)

* The Coastal Dunes; dominating the area to the east of the Nile Delta. The groundwater is generally obtained from shallow hand dug wells and from open hollows, and the daily extraction is about 10,000m3. Expected long term extraction can be increased to 100,000 m3/day (35.0 million m3/year).

* The Local Structural Basins; taking the form of synclinal features and faulted blocks and having wide geographical distribution along the Red Sea coast as well as along the Mediterranean coast. These are best represented by Fuka Basin to the west of Alexandria, Araba Basin to the south of Suez, Ras Gharib Basin to the west of Gharib Oil Field and including Shagar Water

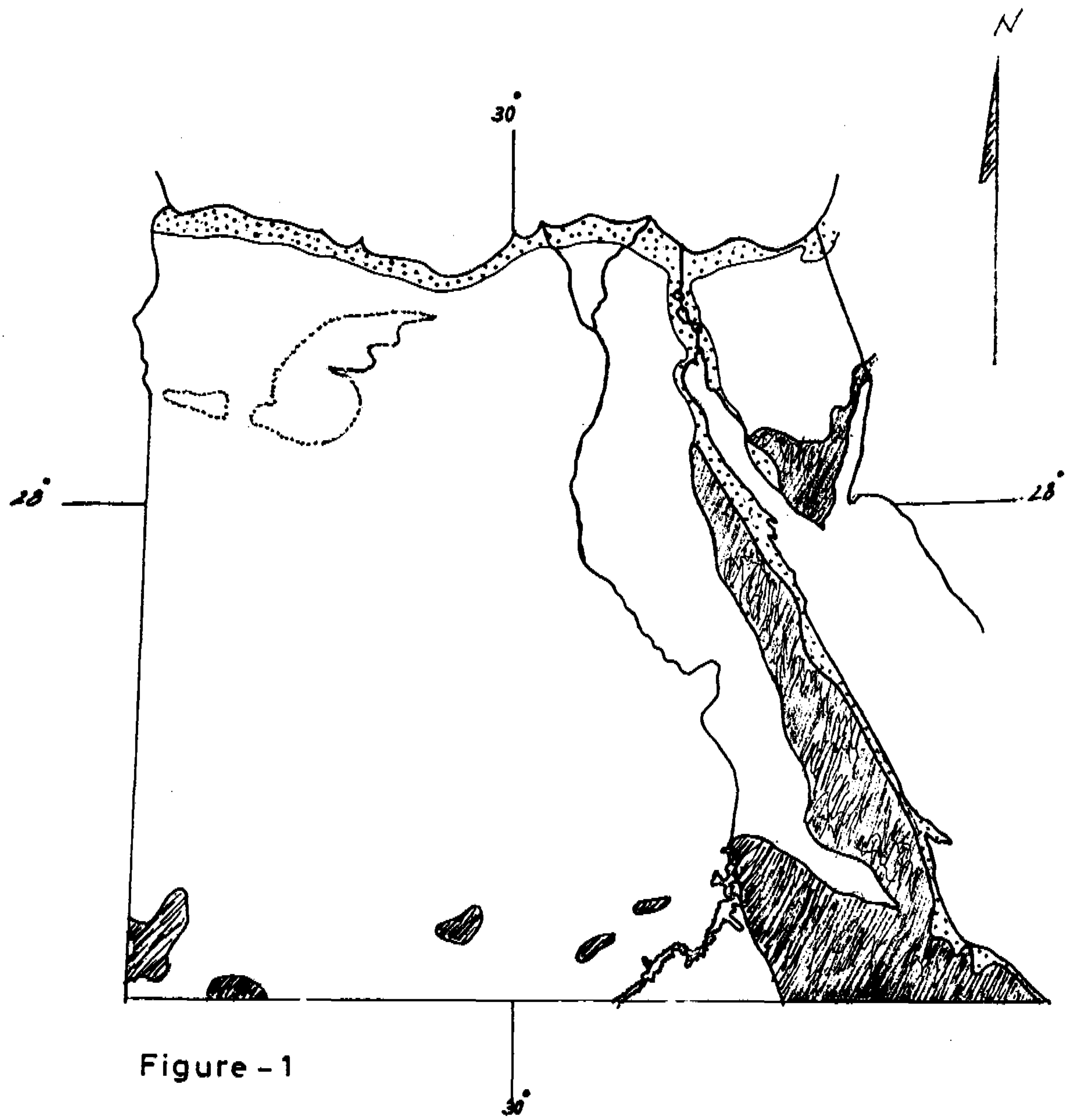

Figure - 1

COASTAL AQUIFER SYSTEMS

Fields, Esh Mallaha Basin to the west of Hurghada, West Quseir Basins Bitter Lakes - Ayoun Musa Basin, to the southeast of Suez and El-Qaa Synclinal Basin to the north of El-Tor in West Sinai. The daily extraction of groundwater from such basinal areas is of the order 50,000 m3 and the estimated long term withdrawal is about 150,000 m3/d (50 million m3/year).

* The Fluvial and Fluvio-Marine Deposits; in the deltaic portions of the old rivers (Wadis) , which terminate into the Mediterranean Sea, the Res Sea proper and the Gulf areas. These are best represented by Wadi El-Arish and other adjacent wadis in North Sinai, Wadi Dara to the west of Hurghada, Wadi Ethil to the south of Quseir, Wadi Qalalat to the south of Berenice, Wadi Hodein to the west of Halaib, Wadi Sudr - Wadi Wardan Wadi Gharandel in West Sinai, Wadi Baba-Wadi Sidri-Wadi Feiran also in West Sinai and at least five main wadis draining into the Gulf of Aqaba. The daily groundwater extraction in such deltaic areas is less than 100,000 m3, and the possible long term production is expected to reach 200,000 m3/day (70.0 million m3/year) These are distributed geographically as follows :

- Mediterranean 100,000 m3/day,
- West Coast Gulf of Suez and Red Sea 30,000 m3/day,
- East Coast Gulf Suez 50,000 m3/day and
- Gulf of Aqaba 20,000 m3/day.

4.2 The Nile And Delta Fringes

The fringes of the Nile -Delta Basin in Egypt, occupying an area of about 25,000 km2, are underlain by low saline water (2000 ppm to 3000 ppm). The chemical quality of the water is generally sodium- magnesium chloride, mixed occassionally with sodium bicarbonate. The thickness of the saturated zone is in excess of 50m and constitutes a fraction of the complex Nile Aquifer System. This water is renewable, easily exploitable and is recharged from different sources including:-

- the excess irrigation water in the new reclamation areas,
- the lateral seepages from the adjacent water bearing layers, and
- the vertical upward leakages from the underlying high pressure water in the Nubian Sandstone Aquifer System.

The large reservoir, which extends, in part, below the fresh groundwater reservoir in the Nile basinal area has not been explored in any acceptable deltails. The reserve estimates of the low saline water in that basin can be of the order of 400 milliard m3. The geographical distribution comprises, essentially, the following (Figuer - 2)

- the area north of the Nile Delta, including the lowland areas,
- the area east of the Nile Delta, including El Mullak-El Salhiya,
- the area west of the Nile Delta, including El-Tahreer and West El-Nubaria,
- the fringes of the Nile Valley in Middle Egypt, including El-Khofug areas and El Saff and,
- the fringes of the Nile Valley in Upper Egypt, including the new reclamation areas at Assuit, Sohag, Qena, Esna and Kom Ombo.

The daily extraction is at present increasing very rapidly and almost without control. This amounts, roughly, to 2.0 million m3/d (700.0 million m3/y). Assuming an area of 4500 km2 of arable lands (1.0 million feddans) and assuming an irrigathion demand of 5000 m3/year/ feddan, the long term annual extraction will reach 5.0 milliard m3 (14.0 million m3/day) in the coming century .

4.3 The Ancient Delta

That deltaic area ($\pm$ 10,000 Km2) occupies a portion of the extensive gravelly desert located to the south and east of the Qattara Depression. It is underlain by a common aquifer system in the Oligo-Miocene rocks, which is known as El-Moghra (El-Moghra is a deserted oasis located at the eastern tip of the Qattara Depression). The groundwater in that aquifer system is nonrenewable and is generally brackish to saline. There is only a small wedge of fresh water in the eastern portion of that deltaic area , which is located adjacent to the present Nile Delta (Figure - 3). In this groundwater reservoir, the storage capacity is great and can be roughly estimated as 100 milliard m3. The eastern portion of that aquifer is under extensive exploitation and the daily extraction is of the order of 0.3 million m3 (100 million m3/y), there are indications of the depletion of the aquifer and a deterioration of the quality of the water. It is expected that the daily extraction of the low saline water will reach 0.5 million m3 in the coming decades.

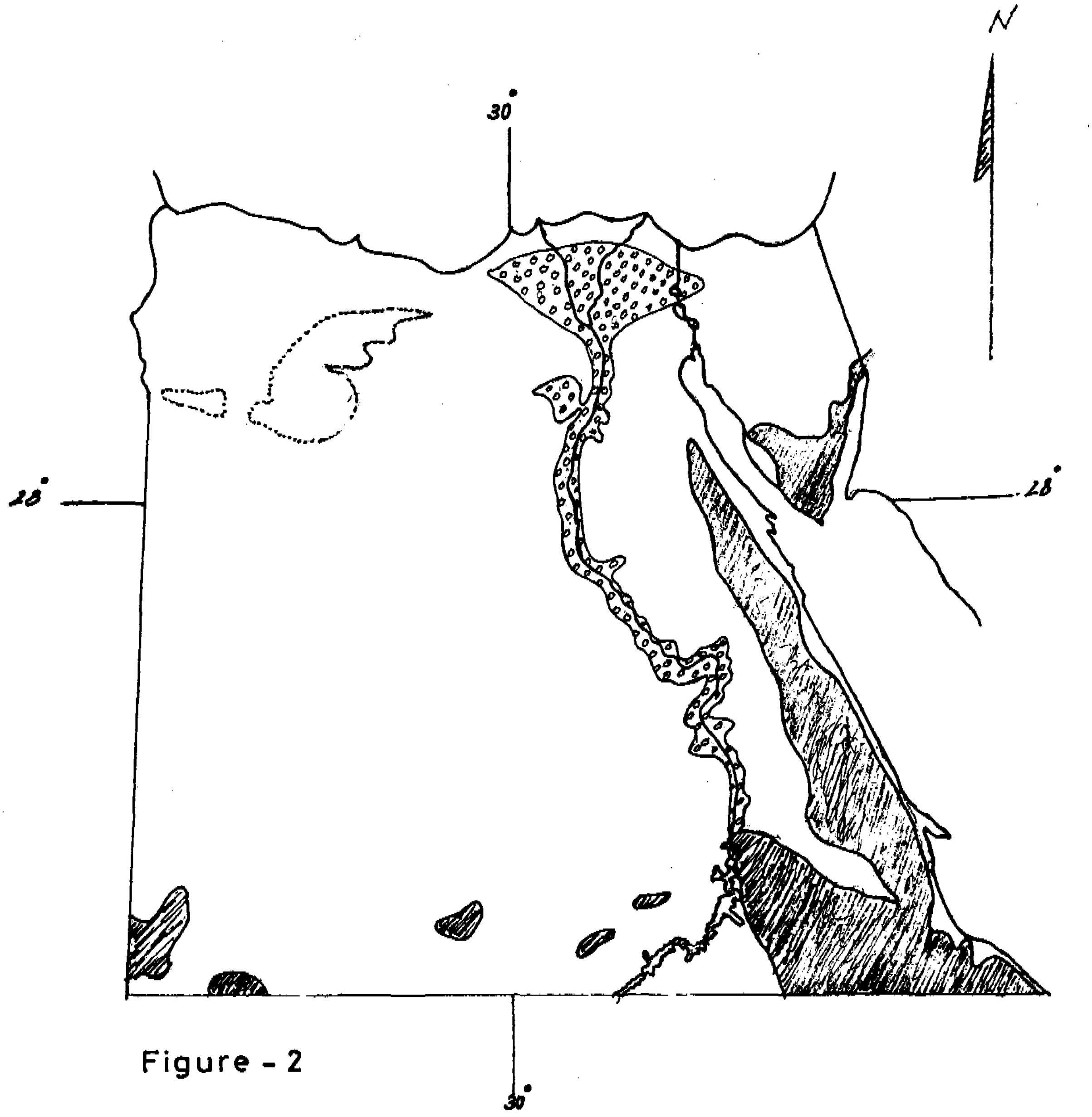

Figure - 2

NILE- DELTA AQUIFER SYSTEMS

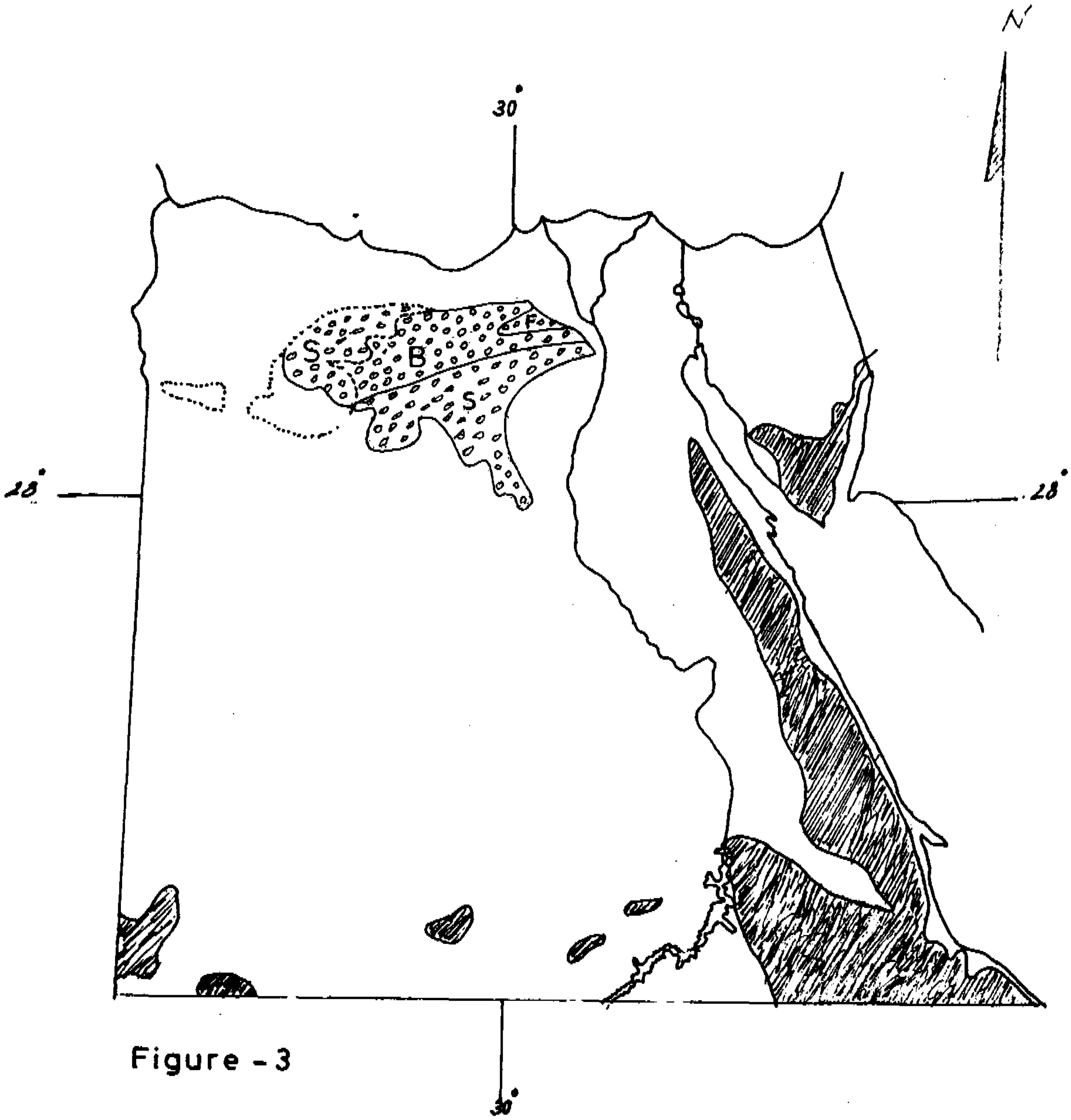

Figure - 3

ANCIENT- DELTA AQUIFER SYSTEMS

4.4 The Periferies Of The Nubian Sandstone Reservoir

This reservoir, underlying much of the desert area west of the Nile, is bounded on the eastern side as well as on the northern side by an extensive belt, where brackish groundwater is available. This type of water is reported essentially in the subsurface, in oil and water wells drilled in various localities, including north Sewa Oasis, north Qara Oasis (to the west of the Qattara depression), northeast Bahariya Oasis, between Assuit and Wadi El- Allaqui in the Nile valley and in several localities in the Eastern Desert and the Sinai Peninsula (Figure - 4).

The reserve estimates of low saline groundwater in that belt is more than 1000 milliard m3 and the water flows naturally and / or from wells to the surface in the Nile Valley area to the south of Assuit and also to the west of the Qattara area. The drilling depths vary from 500 to 1000 m and the production rates are low to moderate . It is obvious that the exploitation of this type of water is dependant on the economy, urgency of land reclamation and the accessibility. At present the daily extration of the Nubian Sandstone low saline groundwater is very limited and there are good possibilities to reach the amount of 0.5 to to 1.0 million m3 in the coming century (150 million m3 to 300 million m3 / year).

4.5 The Fissured Carbonates

These form an extensive and multilayered aquifer system in Egypt (Figure 5) , possibly in hydraulic connection with the underlying high pressure Nubian Sandstone Aquifer System . The water is generally brackish to saline. This aquifer is the source of water issuing from several hundred springs located mainly to the south and to the west of the Qattara Depression. It is exploited on a small scale in Sinai. The reserve estimates are in excess of 100 milliard m3 and the daily output (either naturally or from drilled wells) is in excess of 0.5 million m3 (200.0 million m3 / y). There are potential locations in the west Qattara area for the development of the carbonate aquifer system and it is expected that more than one million m3 of low saline water will be in daily use in the coming century (0.5 milliard m3/y).

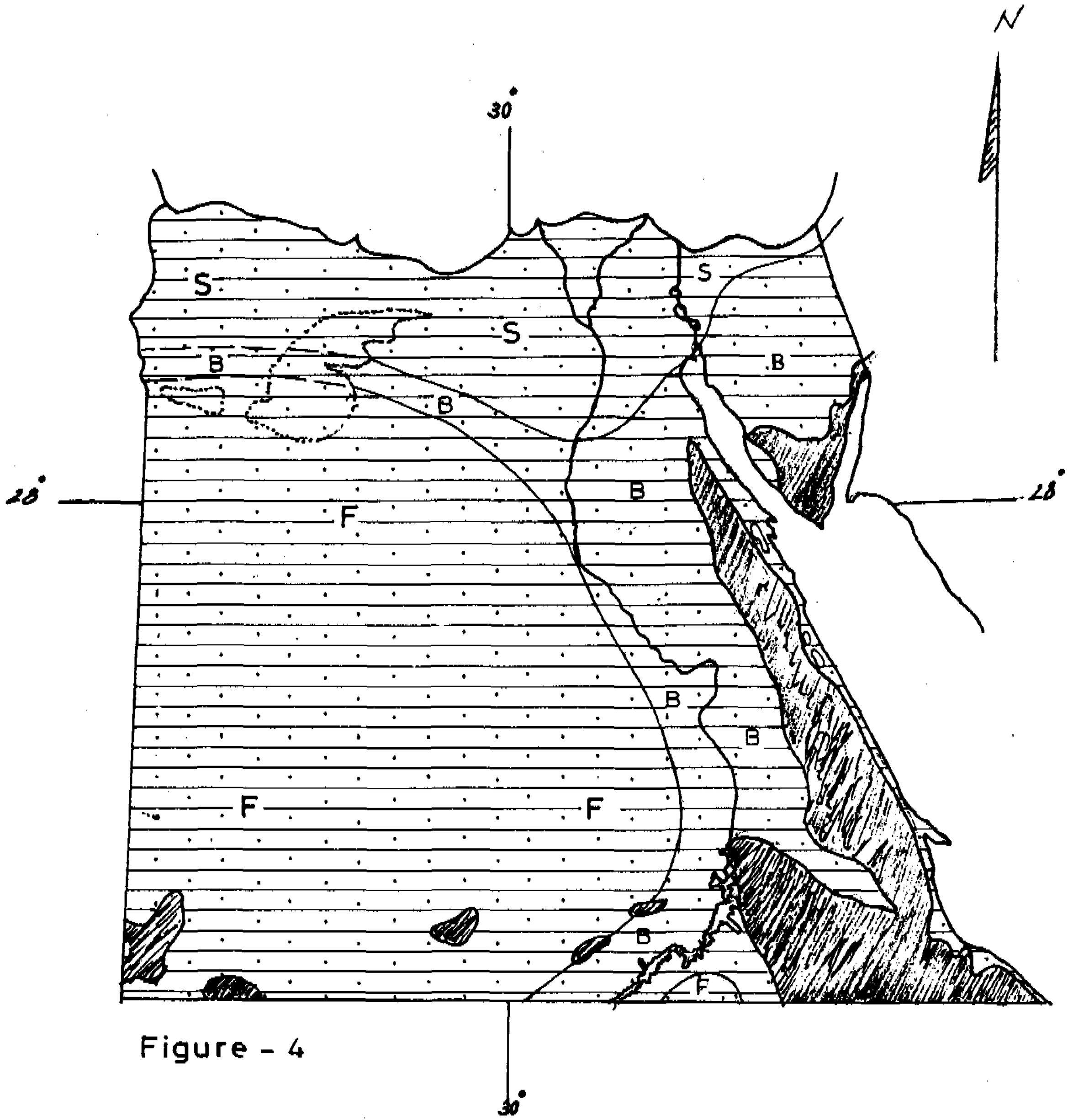

Figure - 4

NUBIAN SANDSTONE AQUIFER SYSTEMS

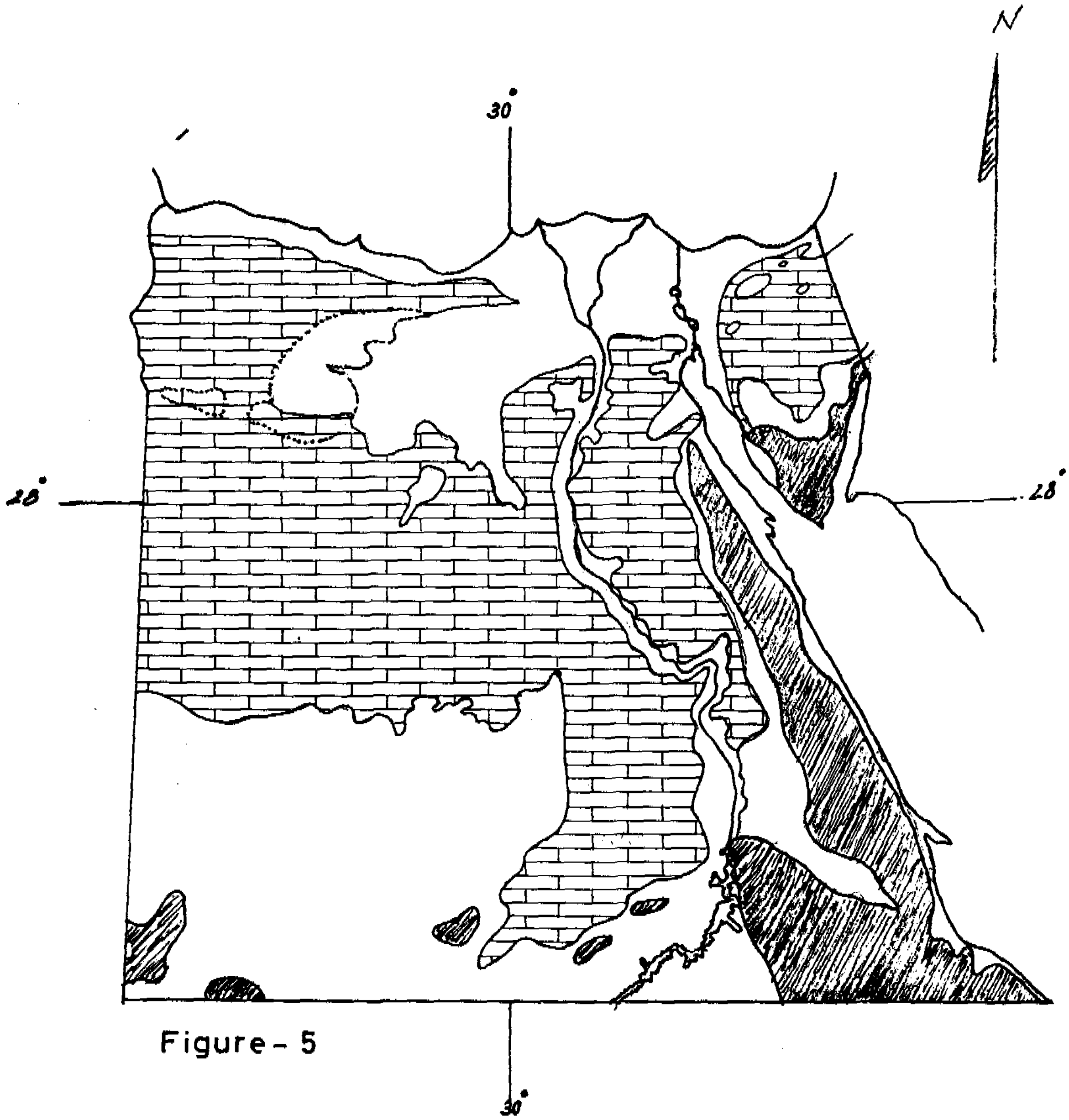

Figure-5

CARBONATE AQUIFER SYSTEMS

5- IRRIGATION DEMANDS OF LOW SALINE GROUNDWATER

In view of the discussion given above, it appears that areas which can be recommended for early action to use low saline water in non-traditonal agriculture, comprise the whole Mediterranean Littoral Zone ($\pm$ 4,000 km2) portions of the Red Sea Littoral Zone ($\pm$ 1000 km2) and the fringes of the Nile-Delta areas. In such areas the potential resources are :-

* the wide occurrences of patchy areas covered with moderately thick alluvial soils. Such areas, which are described as good to poorly arable lands have different degrees of salinity. The areal extent is of the order of 1.0 million feddans, and
* the proven reserves of renewable low saline groundwater amounting to 200 milliard m3. The yearly extraction is estimated as 200 million m3 (only 0.001 %). This amount can be increased, progressively, to reach 5.0 milliard m3.

In the coastal desert areas, the experimental research work of the past 20 years, carried out by the Desert Research Center and ACSAD (1977) and by Mahdia (verbal communication (1994) comprises :

* the use of water with different salinity values, untill the recommended level of 4000 ppm is reached,
* the use of growth regulators, and
* the determination of the best irrigation demands, which is 25 % higher than the field capacity. This capacity, amounting to 5000 m3 per feddan per year, is little modified according to the soil type and to the nature of the crop.

The development of available resources of low saline water in Egypt for the sustainable land utilization needs that special attention should be given to the environmental concern, especially as regards the salinization of the adjacent arable land areas. This can be achieved by :-

- the involvement of different adminstrative and technical levels and
- the training of the water and land users.

This necessitates that the Desert Research Center must take immediate action to integrate efforts with other departments, including the Water Research Center to minimize constraints resulting from the use of low saline water in the reclamation of at least one million feddan both in the fringes of the Nile and Delta and in coastal deserts of Egypt.

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MEETING FUTURE WATER DEMANDS IN PALESTINE THROUGH POPULATION RE-DISTRIBUTION

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ABSTRACT: Following the recent political developments and the Palestinian-Israeli agreement, then it is expected that about eight hundred thousands of Palestinians which were forced out of the West Bank and Gaza Strip (referred to as *Palestine in this paper*) after the 1967 war will return back to their homes within the next five years. An additional number of Palestinians who left Palestine after the 1948 war will come back after the next five years.

Population density in the Gaza Strip represent one of the highest in the world, while in many areas in the West Bank the density is low. Therefore, redistribution of population, is becoming increasingly essential, as readily available water resources are exhausted and surplus water becomes more expensive to develop.

The water problem in the Gaza Strip consist of that the water supply or withdrawal (about 112 mcm/yr) is double the annual recharge capacity of the aquifer (about 50 to 60 mcm/yr). The redistribution of about 260000 Palestinians from the Gaza Strip to the eastern slopes of the West Bank along with good water resources management plan including comprehensive wastewater treatment and reuse, water harvesting techniques, and small desalination units, the future water demand for Palestine until 2010 can be met and more economic and social prosperity and stability in the region can be created.

PREFACE

Palestine is an Arab country which is located in the center of the area connecting the three continents; Asia, Africa, and Europe. It is bound by the Mediterranean sea in the west, by Jordan and Syria in the east, by Lebanon in the north, and by the Sinai Peninsula in the south as shown in Figure 1.

The partition plan for Palestine, outlined in the UN General Assembly Resolution 181 of November 1947 called for the establishment of an Arab state and a Jewish state in Palestine. The borders of the two states were arranged in a way which allowed for direct contact between all parts of each state. However, the actual borders of the Jewish State of Israel as declared in 1948 exceeded the borders outlined in the partition plan, and all contact between what is now known as the West Bank and the Gaza Strip was completely severed. The West Bank became part of Jordan and the Gaza Strip came under Egyptian military supervision.

Following the recent political developments in the Middle East ; the Peace Conference on the Middle East launched in Madrid in November 1991; and the signing of the Palestinian-Israeli Declaration of Principles (DOP) and the agreement for its implementation, then the West Bank and the Gaza Strip will constitute a new Palestinian entity (geographic and political) which will refer to as Palestine in this paper. Accordingly, Israel and Palestine together will constitute Historic Palestine (see Figure 1).

INTRODUCTION

Meeting future water demands in Palestine is of high importance to the newly formed Palestinian National Authority (PNA). Although water demand varies with organization of the system, land use, and socio-economic conditions, overpopulation and the noncompliance between population and water resources represent serious constraining factors not only to the water demand but also for the sustainable development in Palestine.

The present and future rapid increase of water demands in Palestine are due to the accelerating rates of reconstruction, construction and development in addition to the expected lifting of the constraints imposed by the Israeli military authorities on Palestinian water use during the last 27 years.

The rapid increase in population is another important factor influencing future water demand in Palestine. In addition to the rapid natural growth, a forced increase in population is expected due to the return of Palestinian people from the diaspora.

Population density in the Gaza Strip represent one of the highest in the world, while in many areas in the West Bank the density is low, specially the eastern slopes (see Figure 2). Therefore, redistribution of population, is becoming increasingly essential, as readily available water resources are exhausted and surplus water becomes more expensive to develop.

The present and growing gap between water supply and demand in Palestine holds considerable potential for internal political, economic, and social instability and consequently may damage the ongoing peace process.

The paper will discuss the water problem in Palestine, water demand projections under various scenarios, distribution of population and water resources, and meeting future water demands through population distribution.

STUDY AREA

The West Bank, with an area of 5572 square kilometers extending for approximately 155 kilometers in length and 60 kilometers in width, is located mainly in the mountainous lay, but it contains the section of the Jordan valley between the Beisan valley in the north and the Dead sea in the south and small areas in the coastal plains near Tulkarm and Qalqilia[1,2,3].

The Gaza Strip, with an area of 367 square kilometers extending for approximately 41 kilometers in length and approximately 7 to 9 kilometers in width, is situated in the southern part of the coastal plains.

Palestine is longitudinally divided into three main topographical regions running roughly north-south: the coastal plains along the Mediterranean in the west, the mountainous lay in the center, and the Jordan Rift valley in the east. The coastal plain region elevation is about 5 meters above mean sea level, with a line of sand dunes and sandstone hills along the sea shore and lower-lying heavier soils further inland. The transition from the coastal plains to the central mountainous region is sharp especially in the northern part. South of the Jerusalem latitude the transition occurs through an interim area of low foothills. The distance between the axis of the mountainous region and the Jordan river is about 30 kilometers.

The Jordan Valley is the northernmost part of the Syrio-African Rift Valley. In the it was a plateau connecting lake Tiberias with the Dead sea. The base of the Valley is composed of marl layers originating from the Neogene and early Pleistocene period. Mainly among the western margins of the Valley, silt brought down during floods has accumulated. The Valley gets progressively wider from north to south. It has two terraces; the flood plain of the Jordan River, called Zhor, and the rest of the Valley, called Ghor.

Most of the soils in the West Bank are light grey to grayish brown soils. These soils are not particularly fertile but they may be enriched by manure and chemical fertilizers. In the Gaza Strip, besides the barren dune belt along the shore, are the coarse-grained, hamra soils, which are adaptable to farming because of the fine textured cover of mineral on each grain.

The climate in Palestine follows the Mediterranean type. There are two clearly defined climatic seasons, a wet winter and a dry hot summer. The rainy season extends from November to April with the lowest temperatures occurring in January and February and with maximum rainfall in January.

The average total annual rainfall for the West Bank is 409 mm and for the Gaza Strip is 275mm. The annual average rainfall levels in the West Bank and for the coastal plains, the mountainous lay and the Jordan Valley are 500-600 mm, 700 mm and 150 mm, respectively. Annual average rainfall in the Gaza Strip varies from about 400 mm in the northern part to about 200 mm in the southern part. Figure 3 illustrate the rainfall distribution in Palestine.

The annual average temperature for the western plains of the West Bank is 19°C while it is 17°C for the mountainous lay and 25°C for the Jordan Valley. The average annual temperature in the Gaza Strip is about 21°C.

POPULATION DISTRIBUTION IN PALESTINE

1. Natural Equation of Population and Natural Resources

Historic Palestine was all over recorded history with Palestinian people living in it[4]. The natural increase of Palestinian population was not reported to alter the equation of resources- population. The balanced equation was not only altered but reversed beginning at the end of the 19th century and continuing until present time when jews from all over the world began to immigrate to Historic Palestine, creating the State of Israel in 1948, and forcing native Palestinians out of their homeland[5].

This un-natural and continuous increase in population in Historic Palestine was not followed or accompanied by an increase in available water resources in the area. No one of the Jewish newcomers to Historic Palestine brought with him a piece of land or a gallon of water. Unlimited increase of inhabitants of an area with scarce resources certainly will result in resource depletion which is the case of many areas in Historic Palestine.

Demographic trends in the West Bank and the Gaza Strip have been closely related to the political developments in the region. After the 1947-1949 war, approximately 372000 Palestinian refugees were forced out of their original places of residence and had to be accommodated in the West Bank, thus increasing its population from about 250000 to a total of about 622000, an instant increase of about 150%. In addition, about 200000 Palestinian refugees were accommodated in the Gaza Strip, increasing its population from 51000 to about 251000, an increase of about 400%. In the few months after the 1967 war more than 200000 Palestinians left the West Bank and the Gaza Strip to Jordan and Egypt[6].

2. Present Status of Population Distribution in Palestine

The present, 1994, total population of Palestine, the West Bank including East Jerusalem and the Gaza Strip is estimated at approximately 2.11 millions. Table 1 is a summary of population and water resources distribution in Palestine.

The population growth rate in the West Bank and the Gaza Strip for the last five years ranged from 3.2% to 3.5%. The Palestinian people of the West Bank and the Gaza Strip are generally young. About 80% of the West Bank population and 75% of the Gaza Strip population are below 35 years of age. This fact, coupled with the restriction on economic development and land use imposed by the Israeli occupation in the last 27 years, and the absence of major local investment, resulted in a noncompliance between population and resources including water resources.

The populations of the West Bank and the Gaza Strip are distributed by area of living as follows: In the West Bank 24%- 27% of the population lived in urban area, 63%-67% in rural areas, and 7%-10% in refugee camps. In the Gaza Strip 44%-50% of the population lived in urban areas, 9%-11% in rural areas, and 40%-46% in refugee camps.

Population density in the West Bank ranges from 71 to 609 persons/km² with an overall average of 146 persons/km² (see Table 1). There is a geographic trend in the distribution of population density which is as you move from west toward east population density decreases (eg. Tulkarem 367 in the west to Nablus 187 in the center to Jericho 71 persons/km² in the east). As shown in Table 1, population density in the Jerusalem district is the highest in the West Bank. Another characteristic of the population distribution in the West Bank is that the population residing in the northern districts of Nablus, Jenin, and Tulkarem represent about 45% of the total population of the West Bank.

The population density in the Gaza Strip ranges between 1543 in the southern parts to 3700 in the northern parts with an overall average of 2374 persons/km².

AVAILABLE WATER RESOURCES

Ground water constitutes the major source for water in the OPT's. The ground water recharge source in the West Bank is the direct infiltration of rainwater through fractured rocks and porous soil. The estimated recharge is about 648 mcm/yr[7], while the total water extraction is 115 mcm/yr. This seems to indicate that the consumption is far less than the recharge and that there is a surplus in ground water. However, the extraction of ground water is influenced by the Israelis who extract almost all the available surplus of 533 mcm/yr to serve the Israeli colonies (about 50 mcm/yr) and Israel (about 483 mcm/yr).

The only surface water source in Palestine is the Jordan river and itstributaries. In Johnston plan, the Palestinian share in water was considered as part of the Jordanian share (774 mcm/yr), the western Ghore canal was proposed to irrigate 160,000 dunams in the western Ghore with 257 mcm/yr and the eastern to irrigate 353,00 dunams with 517 mcm/yr. Thus the proposed, by Johnston plan, Jordanian and Palestinian share would be 517 and 257 mcm/yr, respectively.

Table 1. Population and Water Resources Distribution in Palestine

District	Area	Population		W. Availa.	
		T	Den	T	/c
West Bank					
Northern	129	478	3700		
Nablus	1587	297	187	53.0	178
Tulkarem	332	122	367	166.2	1362
Jenin	572	142	248	112.6	793
Central	72	136	1892		
Jerusalem	284	173	609	4.8	28
Ramallah	770	167	217	173.0	1036
Bethlehem	565	104	184	66.3	638
Jericho	338	24	71	22.9	954
Southern	163	251	1543		
Hebron	1056	216	205	62.2	288
Total	5572	1245	146	661.0	530
Gaza Strip					
Northern	129	478	3700	40.0	84
Gaza City					
Jabalia					
Beit Hanun					
Nazla					
Beit Lahia					
Central	72	136	1892	11.0	81
Nusseirat					
Bureij					
Magazi					
Zuweida					
Deir Balah					
Khan Yunis					
Karara					
Abasan					
Khizaa					
Southern	163	251	1543	9.0	36
Rafah					
Tal Sultan					
Total	367	865	2374	60.0	69
Palestine Grand Total	5939	2110	355	721.0	342

* Does not include the Palestinian share in the Jordan Basin.

The average annual ground water recharge in the Gaza Strip is about 50 to 60 mcm/yr. Ground water withdrawal in the Gaza Strip Is ranging between 104 and 128 mcm/yr with an average of about 112mcm/yr[8]. This makes the annual water deficit in the Gaza Strip of about 52 to 62 mcm.

EXISTING WATER SUPPLY SYSTEM

As of 1991 the percentage of served population in the West Bank equals to or less than 79.6%. However, about 227 thousands people are left unserved[9]. The served population are receiving water intermittently twice a week(see Figure 4).

Municipal water distribution systems in Palestine are either old and deteriorated or relatively new but badly installed, operated, or maintained. The quality of served water is not monitored. Unaccounted for water in municipal water distribution systems range from 30% to 60% of the volume pumped in the system. These high loss rates are due mainly to the deterioration of old mains (some pipes are in use since the thirties), bad operation and maintenance practices, consumer's mal practices, and inaccurate water meters[10].

There are 314 water wells now in operation in the West Bank with an output of 37.9 mcm/yr[11]. Twenty of the 314 wells with total capacity of 14.8 mcm/yr are used for domestic purposes while the rest, 294, are used in agriculture with a capacity of about 23.1 mcm/yr[12]. Although there are over 300 springs in the West Bank, only sixty of them are reliable and used in irrigation with a total capacity of 40 to 50 mcm/yr[13].

The total Palestinian water consumption for domestic purposes in the Gaza Strip is about 27 mcm/yr[14]. Water withdrawal for irrigation purposes is estimated about 60 to 90 mcm/yr. The total irrigated area is about 96.28 thousand dunams of which 33.64 thousand dunams in vegetative irrigation while 62.64 thousand dunams in citrus tree orchards. The total number of water wells drilled for this purpose is about 1936[15].

The water withdrawn by Israeli Jewish settlements in the Gaza Strip is not clear. While Israeli sources claim that settlements water supply is made through 30 to 40 wells drilled in the south western parts of the Gaza Strip with a capacity of 14 to 28 mcm/yr[16].

PROJECTIONS OF FUTURE WATER DEMANDS

In the absence of a definite and accepted regional development plan for Palestine, it is necessary to make some assumptions regarding the possible trends on which the sectoral and regional development planning can be based. For the purpose of this paper, future water demands in Palestine can be discussed within one scenario such that a Palestinian sovereignty will emerge under which palestinians will have full control over all economic and social resources with returnees are allowed to the territories.

Future domestic, agricultural, industrial, and total water demands for Palestine (the West Bank including East Jerusalem and Gaza Strip) is estimated. The assumptions used in these estimations are given bellow[17]:

- The full soviernity period will begin in 1998 (end of the interim period).

- About 2.0 million Palestinians will return to the West Bank including East Jerusalem (about 1203 thousand returnees at a rate of 80000 annually beginning from 1998 to 2010) and Gaza Strip (751 thousands at a rate of 50000 annually) during the first fifteen years of establishment of the Palestinian state .

- The returning population poses the same age structure, fertility and mortality behavior as the resident population.

- Due to infrastructures rehabilitation and buildup and accelerated rates of urbanization, the annual increase in per capita domestic water consumption of 3% for the whole period.

- Water leakages and losses rates in water distribution systems will be reduced from 35% to 20%.

- Refugee camps will be diminished and their Palestinian inhabitants will be distributed in new towns and surrounding urban areas.

Figure 4 summarizes population forecast and estimates of the total water demand for Palestine until 2010 [18]. Based on the above assumptions the predicted total population of the West Bank and the Gaza Strip for the year 2010 is estimated at 3.20 million based on the natural growth of the present resident population, and at 5.39 million. As shown in Figure 4 the total projected water demand for Palestine; for the assumptions made in this study; by the year 2010 will be about 650 mcm/yr or about 4.4 times the 1990 supply rate. Also this future water demand approximately equal to the total renewable water resources in the West Bank and about 92% of those of Palestine.

MEETING FUTURE WATER DEMANDS AND POPULATION RE-DISTRIBUTION

Meeting future water demand in Palestine under the assumptions described in the previous section and as shown in Figure 4 requires almost all of the available ground water resources in Palestine. Since the majority of the Palestinian water resources are located in the West Bank which still under Israeli military control and mostly exploited by either the Israeli settlers in the West Bank or by the Israelis living in Israel, the dependence on these resources is not guaranteed (the final negotiations didn't concluded yet, the final status of these resources is not defined).

At the same time and with the peace process proceeding, Palestinians are returning from the diaspora in high rates, either in the form of Palestinian police and authority's administrative staff and their families or Palestinians living in Jordan and the those returned from the Gulf States and Saudi arabia. Taking the above circumstances in consideration, and taking in consideration the future of the Palestinian people and their sustainable development, a comprehensive solutions to meet the presently growing and future water demands are needed.

The proposed solution in this paper consists of settling Palestinian returnees and encouraging Palestinians in the highly populated areas in Palestine to settle in areas where the population density is low and the water resources are more available. The areas that fulfil these conditions in Palestine are the eastern slopes of the West Bank for the unpopulated area is and the Gaza Strip for the high densely populated.

Eastern slopes of the West Bank are the most unpopulated area in Palestine (see Figure 2 and Table 1-the district of Jericho) and in addition to the ground water and surface runoff water it is expected to receive the Palestinian share in the Jordan Basin through the Western Ghore canal. However, and in order to evaluate the use of population redistribution as a technique to meeting future water demand in Palestine, some important questions need and are addressed and answered in the following paragraphs:

Why Still Unpopulated?

The reasons behind why the Eastern slopes of the West Bank still unpopulated are summarized bellow:

- the Israeli policies and practices imposed during the last 27 years including annexation and/or closure of Palestinian land, limits and quotas on water use and access and mobility to Palestinian water resources,
- hot climate,
- little if any economic development,
- lack of public services (water and energy supply, transport , communication and telecommunication, education, and health care).
- lack of funding
- and others.

What Actions Need to be Done to make it Feasibly Populated?

In order to make it attractive and acceptable for Palestinians either coming from outside Palestine or from various locations within Palestine to settle in new towns located in the eastern slopes of the West Bank, the following actions need to be done:

- The new towns in the eastern slopes should be planned in an integrated master plan for the entire area and as part of the master plan for Palestine. In this master plan agriculture, agricultural industry, recreation and tourism need to be emphasized.

- Agricultural development of the area should be done through a strong institution (e.g., Jordan Valley Authority) for the whole area. For maximum involvement of people, this institution should be owned by the people of the area through a special ponds policy and run by a publicly elected board.

- For optimal use of available water resources,

    - agriculture should be based on the principle of attaining self sufficiency for the Palestinian economy and market and not for e...

- Non-water related industry and water efficient firms should be planned in the area for job creation such as electronic industry.

- The water services should include wastewater collection, treatment, and full reuse program.

- Prime importance should be given to the appropriate institutional and administrative framework for programs and management of Palestinian water resources.

- All possible alternative sources of water should looked at such as surface and urban storm runoff control projects, desalination of brackish water, various water conservation and harvesting projects, and others.

- A special credit bank should be established in the area for supporting new settlers with special loans policy to agriculture, industry, and tourism.

- Special enterprize zone as Tax Free Zone should be created in the area on the borders with Jordan and Israel.

- Good public services and operation and maintenance departments and teams should be given high priority along with special pricing policy for the first 5-10 years. These services include water, energy, transportation and telecommunications, schools, health, and waste collection and disposal.

- Excellent communication links with other urban centers in Palestine should be considered in the plan and established timely on the ground.

When These Actions Need to be Implemented?

Currently high unemployment and poverty levels are taking place in the Gaza Strip due to limitations and closure of the job-labor market imposed by the Israeli authorities against Gazans. It is also known that salinity levels are very high in the Gaza Strip to the level that few of the wells operating in the Gaza Strip suitable for human use. Therefore, it was judged that immediate actions need to be done to move some of the population other areas to minimize the water withdrawal from the Gaza Strip aquifers (to restore the aquifers) and to relief the pressure on the job market.

Where and How These Actions will be Implemented?

1. Compliance Between Land and Water Resources: The eastern slopes of the West Bank were studied and found that the total area of the Western Ghore is about 500 km^2 and the land which is suitable or can be reclamated for irrigated agriculture there is about 210000 dunams (about 30% of the total area that can be used in irrigated agriculture in the West Bank) divided into the following areas:

Northern Ghore	26800 donums
Biqua valley	18500
Semi-Ghore	20100
Southern Ghore	145500
Total	210900 donums

The available water resources in the eastern slopes were found to be about 431 mcm/yr. A detailed description of water sources and quantities in the eastern slopes of the West Bank are given in Table 2.

Table 2 Water Resources in the Eastern Slopes

Source		Quantity in mcm/yr
Springs		37
Possible Harvesting reservoirs		
Wadi Fara	4.5	
Wadi Auga	2.3	
Wadi Ahmar	3.5	
Wadi malih	0.75	
Wadi Quilt	1.0	
Total		12
Ground Water Aquifers		125
Jordan River*		257
Grand Total		431

* According to Johnston plan

2. Water Demand Needed: After studying the land area that can be cultivated in the eastern slopes (210000 donums), the following distribution of crops were found in order to satisfy the local needs of the Palestinian needs of Bananas, grapes, palms, and vegetables until 2010:

20000 Donums bananas	40 MCM
20000 Donums Grapes	21 MCM
10000 Donums Palm trees	15 MCM
160000 Donums Vegetables*	192 MCM
Total irrigation demand	268 MCM

* vegetables 1.5 crop a year or 240000 donums a year 800 m^3/season of water,

The human resources associated with this development including needed labor and their families, works needed to cultivate the land, the industrial processes, services in towns, tourism and recreation, and others were found to be about 260 thousands. This number represent about 30% of the present population of the Gaza Strip.

The needed domestic water demand was estimated at 28 mcm/yr assuming that the per capita daily consumption in 2010 equal the present Israeli water demand of 250 liters and that leakages and losses are not more than 15%. Because the size of the industrial demand is not clear for now, 10% of the total demand was preserved for this purpose or 33 mcm/yr. Therefore the total needed water demand for this plan will be 329 mcm/yr (see Table bellow) which is less than the average annual available water resources. The difference was reserved for the domestic water use of some urban areas in the West Bank (especially Nablus and Ramallah) and for variations expected in determining the Palestinian water share in the Jordan basin.

purpose	Demand, mcm/yr
Agricultural demand	268
Industrial demand	33
Domestic demand	28
Total Demand	329

3. Population Re-Distribution and Meeting Total Water Demand: The water problem in the Gaza Strip consist of that the water supply or withdrawal (about 112 mcm/yr) is double the annual recharge capacity of the aquifer (about 50 to 60 mcm/yr). The present water withdrawal in the Gaza Strip is about 112 mcm/yr distributed by purpose of use as follows:

Agricultural Use		75, mcm/yr
Citrus	55	
Vegetables	20	
Domestic Use		27
Jewish settlements		10
Total		122, mcm/yr

If the redistribution plan is to be executed now and a freeze is put on water use in agriculture in the Gaza Strip and transfer the labor working in it to the eastern slopes we save immediately 83 mcm/yr (75 mcm/yr from agriculture and 8 mcm/yr in the domestic sector) of water. Assuming that the Jewish settlements in the Gaza Strip will be dismantled at the end of the final status of the peace process, the total water available in the Gaza Strip will be about 50 to 60 mcm/yr and the per capita water use for domestic and industrial purposes will be 150 l/c-d, the available water will serve about 932 thousands of inhabitants which are a little greater than the present population.

With comprehensive wastewater treatment and reuse, water harvesting techniques, and small desalination units, water for domestic and industrial purposes can be secure for the Gaza Strip until 2010.

The water demand for the West Bank in 2010 will be about 3.58 millions. Using the same assumptions used for the per capita domestic and industrial water demand, the demand need will be about 225 mcm/yr. Knowing that the total annual recharge to the western aquifer of the West Bank is about 380 mcm/yr and the present agricultural water demand for the West Bank is about 80-90 mcm/yr, the total water demand for the West Bank will be also satisfied and a long term reserve for the future is reserved. The West Bank will benefit from the redistribution plan by securing its agricultural needs, have better communication links with surrounding areas, and more opportunities will be open in the economic development branch.

CONCLUSIONS

In conclusion it was found that by the population redistribution can be used successfully to meet the future total water demand in Palestine. The redistribution of about 260000 Palestinians from the Gaza Strip to the eastern slopes of the West Bank along with good water resources management plan was satisfactory to meet the future water demand can be met and more economic and social prosperity and stability in the region can be created.

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RAINFALL-RUNOFF WATER HARVESTING PROSPECTS FOR GREATER AMMAN AND JORDAN

Herbert C. Preul[1]

ABSTRACT

This paper presents an evaluation of the prospects for rainfall-runoff water harvesting as a means of increasing water supplies in Greater Amman and the Hashemite Kingdom of Jordan. Rainfall-runoff water harvesting is a small scale water conservation approache for catching/storing rainfalls and certain runoff waters in a localized area, before the waters enter the usual hydrologic cycle. Generally, the methods are small basic impounding concepts as compared with larger scale river dams and reservoirs. Catchment of rainfall drainage from a building roof with tank storage is the commonest practice. This is an age-old practice used to sustain populations mostly in arid areas of the world, but also in areas where water distribution systems are unavailable. In the following text, water harvesting yields are calculated for residential roofs and for the potential from commercial and industrial areas, as of the years 1990 and projected to 2005. The results show that rainfall-runoff water harvesting is an increasingly attractive consideration in arid areas facing acute water supply shortages, for both villages and urban centers.

DES PERSPECTIVES DE LA COLLECTE DES EAUX DE PLUIE ET DE RUISSELLEMENT POUR L'AGGLOMERATION D'AMMAN ET JORDANIE

RÉSUMÉ

Cette communication presente une evaluation des perspectives d'avenir de la collete des eaux de pluie et de ruisellement comme un moyen d'accoitre l'approvisionnement en eau de l'agglomération d'Amman et du Royaume d'Hashemite de Jordanie. La collete des eaux de pluie et de ruissellement est une approche de conservation d'eau à petite échelle pour capter et mettre en réserve localement des eaux de pluie et certaines eaux de ruissellement avant que celles-ci ne rejoignent le cycle normale d'hydrologie. Généralement, ces méthodes sont des coneptes fondamentaux de petit captage en comparaison avec de grands barrages de fleuve ou de grandes retenues d'eau. Le captage de l'eau de pluie a partir d'un toit d'une maison pour la mise en réserve dans une citerne est la pratique la plus courante. C'est une vieille pratique pour supporter des populations dans des régions arides du monde pour la pluspart, mais aussi dans des regions qui n'ont pas de systeme d'adduction en eau. Dans le texte qui suit, des debits de captage pour l'an 1990 et les projections de l'an 2005 sont calculés pour des toits de quartier residentiel, et aussi ceux qui pouront provenir des zones commerciales et industrielles. Les résultats montrent que le captage des eaux de pluie et de ruissellement est une consideration de plus en plus attrayante dans les regions arides où l'approvisionnement en eau fait sérieusement défaut, aussi bien dans les villages que dans les centres urbains.

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1. RAINFALL-RUNOFF WATER HARVESTING (RRWH) POTENTIAL

Figure 1 shows rainfall isohyets on a map of the Kingdom of Jordan. Rainfall ranges from a high of approximately 500 mm/yr in some areas to a low of 50 mm/yr or less in others. Individual housing rain water harvesting is currently practiced in certain areas in the Kingdom of Jordan particularly where connections to water distribution systems are unavailable. People in some town and rural areas collect rain water in cisterns for one-third or more of their domestic usage. A much greater potential for exploitation of rain water harvesting from building roofs is possible in Jordan, particularly in certain urban areas. Rainfall-runoff harvesting is also possible from impervious areas in certain commercial districts, parking lots, and industrial complexes. However, existing densely developed commercial districts would certainly be difficult to retrofit as compared with facilities included in new construction. Generally, it is considered that RRWH waters would be used for secondary purposes, augmenting the basic supply in urban areas having approved water distribution systems. In more remote areas with scarce rainfall, rainfall-runoff water harvesting may constitute a primary water supply.

2. ESTIMATED RRWH YIELDS FOR GREATER AMMAN

Annual rainfall-runoff water harvest yields, **Y**, are calculated using the following:

Y = (CAR)/100 = cu m/yr (CM/yr)

where: **C** = runoff coefficient
A = roof or drainage area, sq m
R = ave. ann. rainfall, mm

Greater Amman has the greatest potential for rainfall-runoff water harvesting, therefore an analysis has been made for the estimated yield from residential roofs for this metropolitan area based on available year 1985 data published in the Greater Amman Planning Report by a Joint Technical Team (1990). Year 1985 data were the most recent data available at the time of the preparation of this paper, therefore RRWH calculations are on this basis, with projections to years 1990 and 2005 as shown in Table 1. Some basic housing statistics for Greater Amman in the year 1985 are listed below:

900,990 population

144,708 households (HH)

141,000 occuppied dwellings

16,000 vacant dwellings

60% dwellings in low-rise apartment buildings

30% dwellings in one/two story villas & houses

10% dwellings in single story buildings

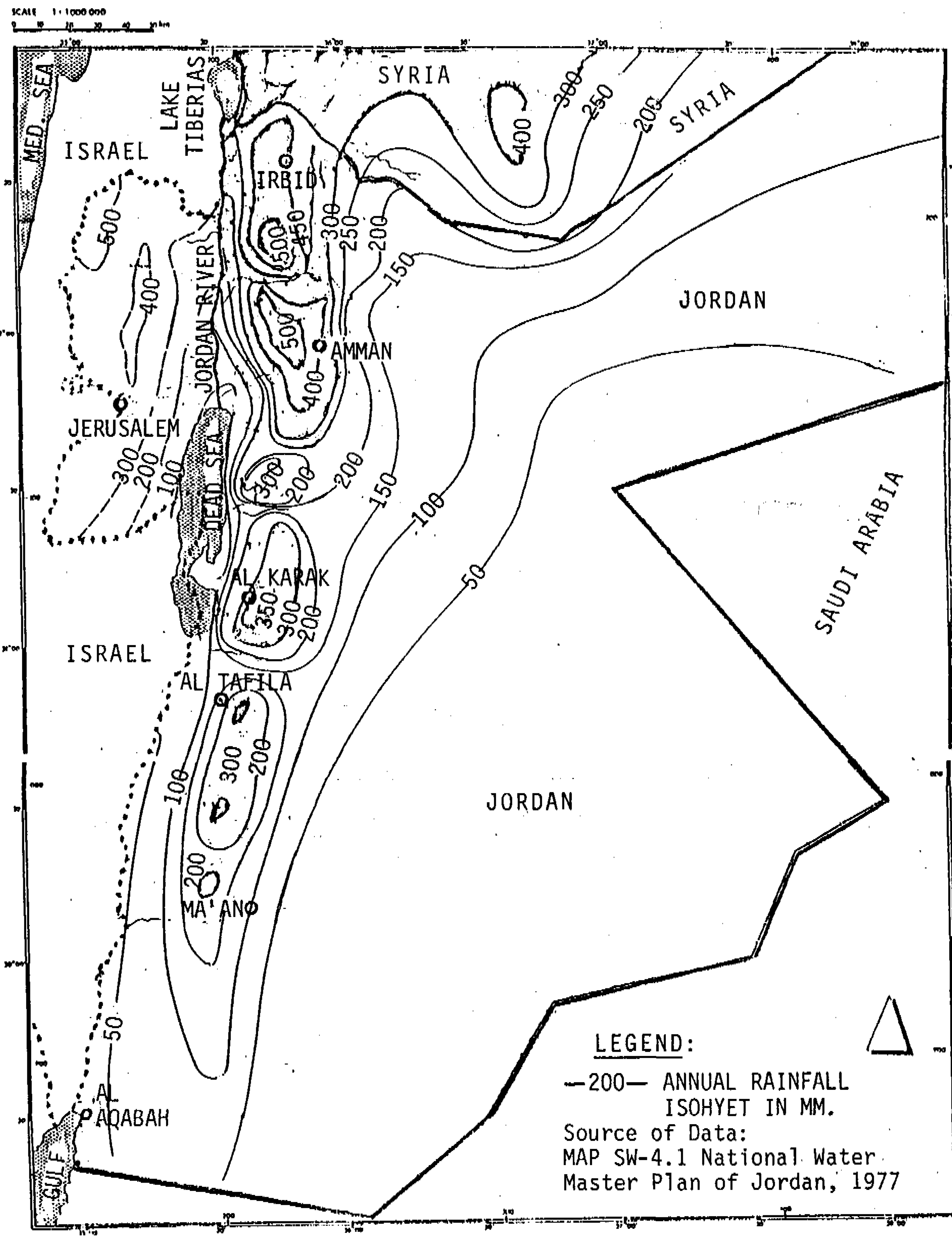

Figure 1 - RAINFALL DISTRIBUTION: ISOHYETS - NORMAL YEAR (Long Term Average) FOR THE HASHEMITE KINGDOM OF JORDAN

For estimating roof areas, the following data were used:

Zone	Households/dunum	Estimated Roof Area
A	1.8 - 2.7 HH/dun	30% of total zone area
B	2.7 - 3.6	35%
C	4.3 - 4.8	40%
D	6.5 - 7.2	44%

Note: 1 dun = 1000 sq m = 0.001 sq km or 1 sq km = 1000 dun
Roof areas estimated considering building construction limits with allowances for streets & sidewalks.

Table 1 - Estimated Gross RRWH Yields from Residential Building Roofs, Greater Amman

Based on 400 mm/yr average rainfall:

Yr 1985 Y= 5,018,700 CM/yr =34.682 CM/HH/yr = 34,682 l/HH/yr

1990 Y= 6,365,000 CM/yr =35.850 CM/HH/yr = 35,850 l/HH/yr

2005 Y=13,068,800 CM/yr =38.052 CM/HH/yr = 38,052 l/HH/yr

Yr 1985 Y= 5,570 l/cap/yr = 15 l/cap/day

1990 Y= 5,760 l/cap/yr = 16 l/cap/day

2005 Y= 6,530 l/cap/yr = 18 l/cap/day

The above Table 1 gross yields increase from year 1985 to 1990 and projected to 2005 due to increases in population and the density of roofs.

3. ESTIMATED RRWH YIELDS FOR KINGDOM OF JORDAN

In 1990, the total population of the Kingdom of Jordan was estimated at 70% urban (10% in refugee camps and informal areas) and 30% rural with the following distribution:

Yr 1990: (urban 2,177,000)+(rural 935,000) = total 3,112,000

Considering an increasing urban population, the following is estimated:

Yr 2005: (urban 3,158,000)+(rural 981,000) = total 4,139,000

There are eight Governorates in the Kingdom with considerable variation in population and average annual rainfall. Applying the yield/cap/yr data developed previously for Greater Amman as a basis, with adjustments for varied average annual rainfalls and populations in the various eight Governorates, the following yield estimates are calculated in Table 2.

Table 2 - Estimated Gross RRWH Yields from Residential Building Roofs for Kingdom of Jordan

Yr 1990: Y =15,093,000 CM/yr =4.85 CM/cap/yr =4,850 l/cap/yr

Yr 2005: Y =23,606,000 CM/yr =5.70 CM/cap/yr =5,700 l/cap/yr

Note that the above per capita yields for the entire Kingdom are lower than in Table 1 for Greater Amman, due to the lower average annual rainfalls in certain of the Governorate areas.

4. HYDROLOGIC TRADE-OFF AND NET WATER CONSERVATION GAINS

It is recognized that the above tabulated gross yields, in both Tables 1 and 2, are an hydrologic trade-off to the extent that these waters would otherwise run off into streets and overland as part of the usual hydrologic cycle. The difference with water harvesting is in catching the rainfall and runoff before it can enter the normal storm water runoff process, thereby avoiding certain losses mainly in the form of evapotranspiration and infiltration. In urban areas, this difference is between an estimated 0.80 runoff factor for roofs and an estimated runoff factor of 0.50 for general urban runoff; resulting in the following water conservation gain:

(0.80 - 0.50)/(0.50) = 60% gain (urban only)

In rural areas, the difference is between an estimated 0.80 runoff factor for roofs and an estimated runoff factor of 0.40 for general rural runoff; resulting in the following water conservation gain:

(0.80 - 0.40)/(0.40) = 100% gain (rural only)

The above runoff coefficients are intended to be conservative for these calculations; actually the roof runoff factor may range up to 0.90 or more and general runoff factors less depending on local conditions.

Table 3 below shows the net water conservation RRWH gains as compared with the normal hydrologic runoff process. The rural areas stand to realize relatively higher gains due to their differing runoff factors.

Table 3 - Net Water Conservation Gains for RRWH from Residential Roofs: Gr. Amman & Kingdon of Jordan

Area	With RRWH	Without RRWH	Net Conserv. Gain	
	Yw	Ywo	Yw - Ywo	Percent
Yr	MCM/yr	MCM/yr	MCM/yr	%

(MCM/yr = million cu. meters/yr)

Greater Amman:				
1990	6.365	3.978	2.387	+60%
2005	13.069	8.168	4.901	+60%
Kingdom of Jordan:				
1990	15.093	8.864	6.229	+70%
2005	23.606	14.075	9.531	+68%

5. RRWH POTENTIAL FROM COMMERCIAL AND INDUSTRIAL AREAS

Further water conservation gains can be realized through rainfall-runoff water harvesting in commercial and industrial areas, including roofs and parking lots. Although more difficult to accomplish in certain areas due to congestion, Table 4 below shows significant possibilities in Greater Amman.

Table 4 - Estimated Gross RRWH Yields from Commercial and Industrial Roofs & Impervious Lots, Greater Amman

Area/District	Total Area	Roofs/Impervious	Y
Year 1990 basis:			
Central Business District	7,800 dun	(50%)=3,900 dun	1.248 MCM/yr
Industrial Areas	16,920 dun	(50%)=8,460 dun	2.707 MCM/yr
Total............			3.955 MCM/yr
Year 2005 basis:			
Total (roughly estimated +10% over 1990)............			4.351 MCM/yr

Considering the same relative runoff factors (C=0.80 with RRWH and C=0.50 without RRWH) as previously used to calculate urban water conservation gains for residential areas, the above totals represent a 60% gain over conditions without RRWH.

6. IMPLEMENTATION AND COSTS

A broad implementation of urban and rural rainfall-runoff water harvesting would require a coordinated program with technical support. Controlling ordinances and architectural regulations would be a starting point. From a water utility cost point of view, such a program becomes attractive because the costs for the complete water harvesting facilities would essentially be borne by the private owners since the systems are individual and not combined into a general water supply distribution system. Existing residential and certain commercial buildings can likely be rather easily retrofitted by directing roof drainage to an appropriately sized tank either above or below ground, as illustrated in Figures 2 and 3.

Storage tanks should be properly sized according to anticipated usage and prevailing average annual rainfall in the area; for ease of handling, tanks may typically be constructed of light weight materials. Fiber-glass tanks are currently manufactured in Amman and are ideal for this purpose. Retrofitting commercial buildings, parking lots, and impervious areas under densely developed conditions would be more of a problem due to the difficulty in positioning storage tanks; however in new construction, this could be a requirement in the design.

Figures 4 and 5 illustrate possible storage system concepts which may be constructed under parking lots and impervious surfaces in commercial areas and industrial complexes. Depending on the positioning of storage tanks, equipment for drawing the waters for use may be comparatively simple: by gravity if above ground from roofs, or with pumps if below ground. Generally, in urban areas where water distribution systems are available, these collected waters would be reserved for secondary use purposes, i.e., garden irrigation, car washing, etc., and would augment the basic potable water supply. However, under certain critical conditions such as in remote rural areas, the quality of roof drainage waters can be up-graded to a potable level through comparatively simple treatment approaches; usually, filtration and disinfection are needed. Commercial and industrial facilities usually have numerous secondary purpose water needs, for which RRWH offers a cost effective supply.

7. SUMMARY AND CONCLUSIONS

The above calculations demonstrate that considerable water conservation gains are possible in Greater Amman and the entire Kingdon of Jordan through rainfall-runoff water harvesting from building roofs, particularly in residential areas. Harvesting in commercial and industrial areas is also possible; industrial complexes are generally prime areas because of their extensive roof and impervious lot areas. As an example for RRWH from Greater Amman residential roofs, the year 1990 estimate in Table 1 shows:

gross yield Y=35,850 l/HouseHold/yr

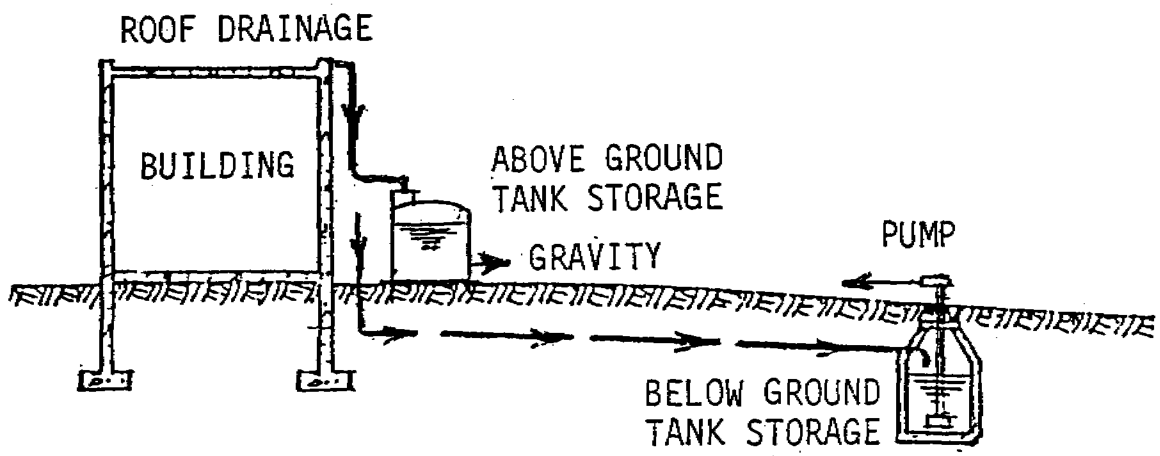

FIGURE 2 - ROOF WATER HARVESTING FACILITY FOR RESIDENTIAL OR COMMERCIAL BUILDINGS

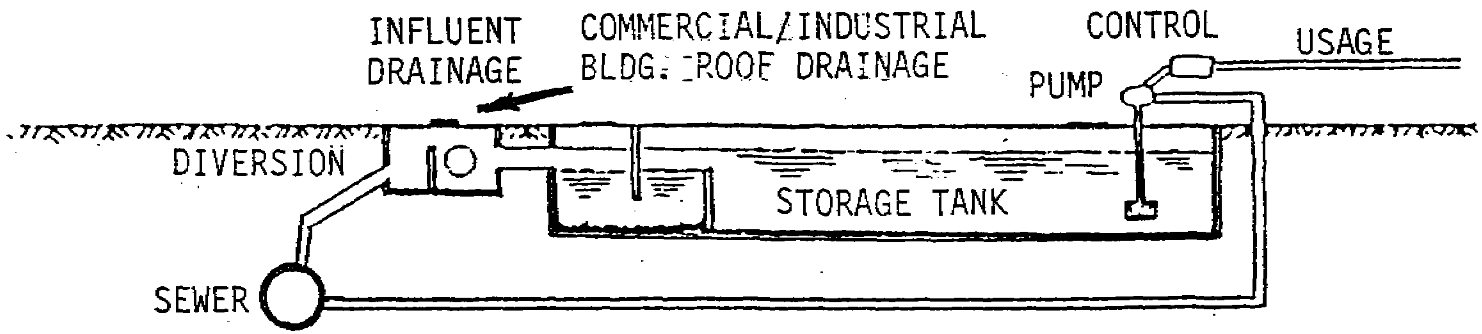

FIGURE 3 - ROOF DRAINAGE HARVESTING FACILITY FOR COMMERCIAL AND INDUSTRIAL BUILDINGS

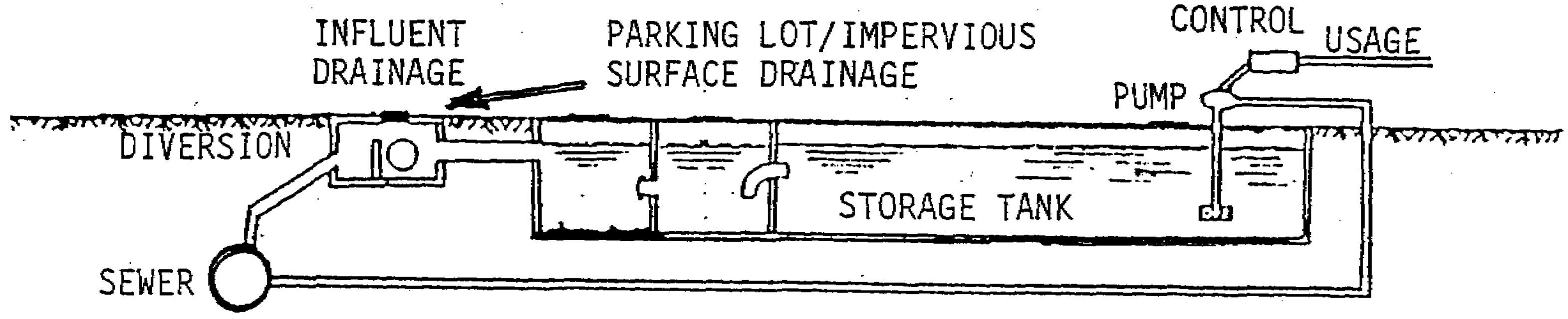

FIGURE 4 - RAINFALL-RUNOFF WATER HARVESTING FACILITY FOR PARKING LOT/IMPERVIOUS SURFACE DRAINAGE

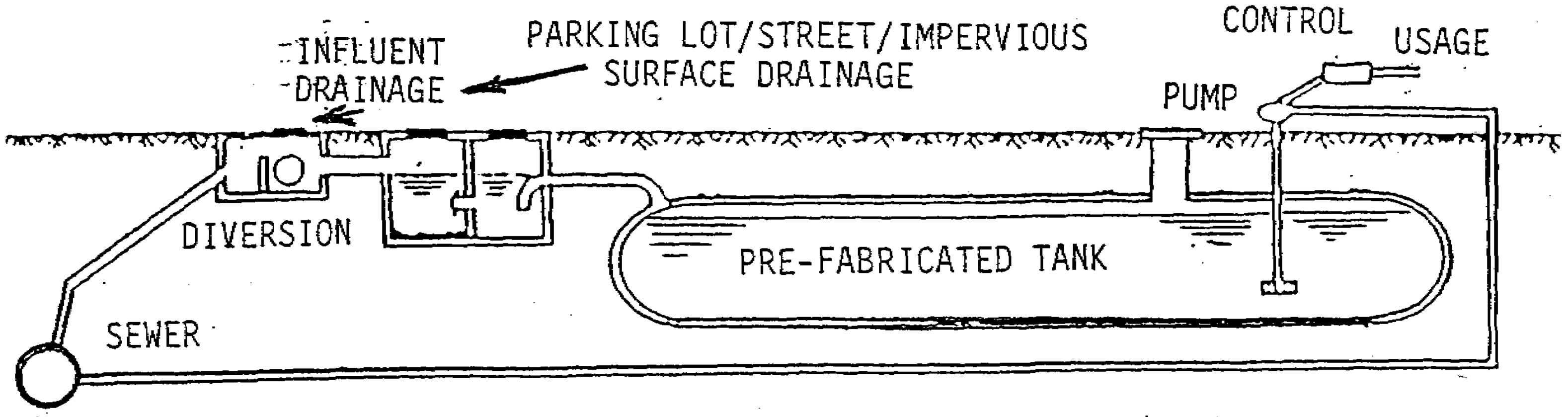

FIGURE 5 - RAINFALL-RUNOFF WATER HARVESTING FACILITY FOR PARKING/STREET/IMPERVIOUS SURFACE DRAINAGE

This translates into 98 l/HH/day or roughly 16 l/cap/day over a one year period, which represents as much as 16+% of an estimated potential domestic demand of 99 l/cap/day (Source: Water Authority of Jordan, 1992).

In towns and villages where domestic usage is generally lower, RRWH from roofs can be expected to provide an even higher percentage of the daily demand. The Water Authority of the Ministry of Water and Irrigation (1992) has reported that in the Governorate of Irbid where house connections are not available, rainwater collection represents up to 32% as a source of water commonly used. In fact, the same report states "Water from the public network is reported to be salty, hard, turbid, over-chlorinated, having taste, of yellow-reddish colour, not usable for even cooking or making tea. As a result of this, the people use rainwater collected in cisterns for drinking and cooking purposes, while the water from the public supply is used for cleaning, washing and plant irrigation."

Or rather than viewing daily usage quantities, another approach for this form of storage is to save it to carry over as a supply during the low rainfall months of the year in Jordan, i.e., May through September, when practically no precipitation occurs and the climate becomes hot and dry. Since the storage is contained in a tank, little evaporation occurs.

Although some technical guidance by water utility organizations is desirable in program implementation, a major advantage of RRWH is that the retrofitting costs are comparatively low and can be done on an individual owner basis.

Under current and future water scarce conditions, water restrictions for secondary usage purposes can be expected especially in urban areas, and water rates will substantially increase. Therefore rainfall-runoff water harvesting becomes an increasingly attractive consideration in arid areas facing acute water supply shortages.

8. ACKNOWLEDGEMENTS

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GROUNDWATER DEVELOPMENT FOR DROUGHT MITIGATION

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ABSTRACT

Aquifers can be considered water conveyance media as well as storage reservoirs. Water can be pumped at any location within the aquifer system irrespective of the recharge location. When aquifers have enough space for water storage, water can be stored during surplus periods and withdrawn during periods of peak water requirements. As such, aquifers can play a considerable role during water drought periods.

In Egypt, renewable aquifers that are believed to play a role in satisfying water requirements for drought mitigation are those belonging to the Nile system. Predictions concerning the possibility of regulating the Nile aquifer system for drought mitigation are carried out with the help of a simulation model on sample regions. Results of this exercise are extended to the remaining regions of the Nile aquifer system; indicating the possibility to withdraw a total of about 9 billion m^3 of groundwater annually from the Nile aquifer system for five successive years, after which pumpage is reduced to half for about three years. The cycle can then be repeated for 8 years period with almost no adverse impacts. This groundwater withdrawal represents 16% of the available Nile surface water.

RÉSUMÉ

Les nappes aquifères peuvent être considérées des tuyaux aussi bien que des résèvoires d'eau. L'eau peut être pompée à n'importe quelle place sans égard à la place de l'alimentation. Quand l'aquifère contient assez d'éspace, l'eau peut être emmagasinée durant les périodes de hautes demandes en eau. Comme cela, les aquifères peuvent jouer un rōle important durant les périodes de sècheresse.

En Egypte, les aquifères qui peuvent jouer un rôle important durant les périodes de sècheresse sont ceux du système de la rivière du Nil. Un modèle a été utilizé pour éxaminer les possibilités différentes. Les résultats de cet exercise indiquent que la quantité d'eau qui peut être pompée sans affecter le système est 9 milliards m^3/année pour 5 années consécutives; et puis le taux sera réduit à moitié pendant les 3 années suivantes. Ce cycle de 8 ans peut être répété sans affecter le reseau. La quantitée d'eau soutérraine obtenue selon ce régime représente 16% du taux de la rivière du Nil.

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1. INTRODUCTION

Egypt is considered a very arid country. The average annual rainfall seldom exceeds 200 mm along the northern and south-eastern coasts; declining rapidly inland, and becoming almost nil south of Cairo. This meager rainfall occurs in the winter in the form of scattered showers, and cannot be dependent upon for extensive agricultural production. Thus, reliable availability of irrigation water is essential for agriculture in the major area of the country.

The main and almost exclusive source of surface water is the River Nile. The average annual flow at Aswan is about 84 billion m^3. The average annual evaporation and other losses in the High Dam Lake are estimated at 10 billion m^3, leaving a net usable annual flow of 74 billion m^3. Under the 1959 treaty, 55.5 billion m^3 are allocated to Egypt, and 18.5 billion m^3 for the Sudan.

The other source of water is groundwater. Groundwater in Egypt can be divided into two categories, the Nile aquifer system and desert fringes irrigated with surface water on one hand, and the remaining deserts, desert fringes, and Sinai on the other hand. The most important aquifers are the Nubian Sandstone, the Plio-Pleistocene, the carbonates, and the coastal aquifers. Groundwater in the Nubian sandstone and Plio-Pleistocene aquifers is mostly fossil water. The carbonate formation, which covers a large area of the Western Desert, is not yet explored and may contain brackish water. The coastal aquifers receive high recharge from winter rainfall; the majority of which is lost to the sea due to lack of proper harvesting.

The Nile aquifer system is almost the only renewable aquifer. It is mainly replenished from irrigation activities. Accordingly, groundwater in this aquifer cannot be considered a source in itself; and the aquifer can be considered a storage reservoir analogous to the High Dam reservoir. The total storage capacity of the Nile aquifer system is estimated at 500 billion m^3. Moreover, its hydraulic characteristics are suitable for the storage and transmission of water.

In this paper, a simulation model is applied on sample areas in the Nile Valley and Delta to test the impact of applying various groundwater development scenarios. The result of the exercise are extended to cover the remaining part of the Nile aquifer system.

2. HYDROGEOLOGIC ENVIRONMENT FOR THE NILE SYSTEM

The Nile enters Egypt at Wadi Halfa, south of Aswan. This area is at present occupied by the High Aswan Lake. From Aswan to Cairo, the River meanders until it reaches Cairo. The total length of the River course between Aswan and Cairo is about 950 km. The Nile enters the Delta depression north of Cairo. At a distance of about 20 km north of Cairo, the River divides into two branches, each of which meanders separately through the Delta to the sea. The western branch, Rosetta, is 239 km long and debouches into the Mediterranean at Rosetta city. The eastern branch, Damietta, is 244 km long and debouches into the Mediterranean at Damietta city. In the Nile Valley and Delta, extensive man-made drainage systems exist especially in the traditionally cultivated old land. Some are extended to the reclaimed areas. Such drainage systems are supplemented with sets of lifting stations, which allow for the disposal of the land drainage water to the River, Mediterranean sea, or to the northern lakes.

The landscape in Egypt can be broadly divided into the elevated structural plateaux and the low plains which include the fluviatile and the coastal plains. These geomorphologic units play a significant role in determining the hydrogeological framework of Egypt. The structural plateaux constitute the active and semi-active watershed areas. The plains contain, in certain areas, very productive aquifers and act in other areas as discharging basins. The narrow coastal plains are characterized by the presence of local Tertiary and Quaternary aquifer systems.

Geologically, Egypt is a portion of the northward overlap of the Nubian Arabian massif. In the south-east part of the country, the basement rocks are exposed and constitute a portion of the African craton. According to most researchers, this craton was formed during the Pre-Cambrian and possibly also during the Cambrian. It is composed of a number of crustal plates or segments separated by major N/NW-S/SE faults (El Shazly, 1966), as presented in Figure 1. Further complications were introduced by folding and wrench faulting.

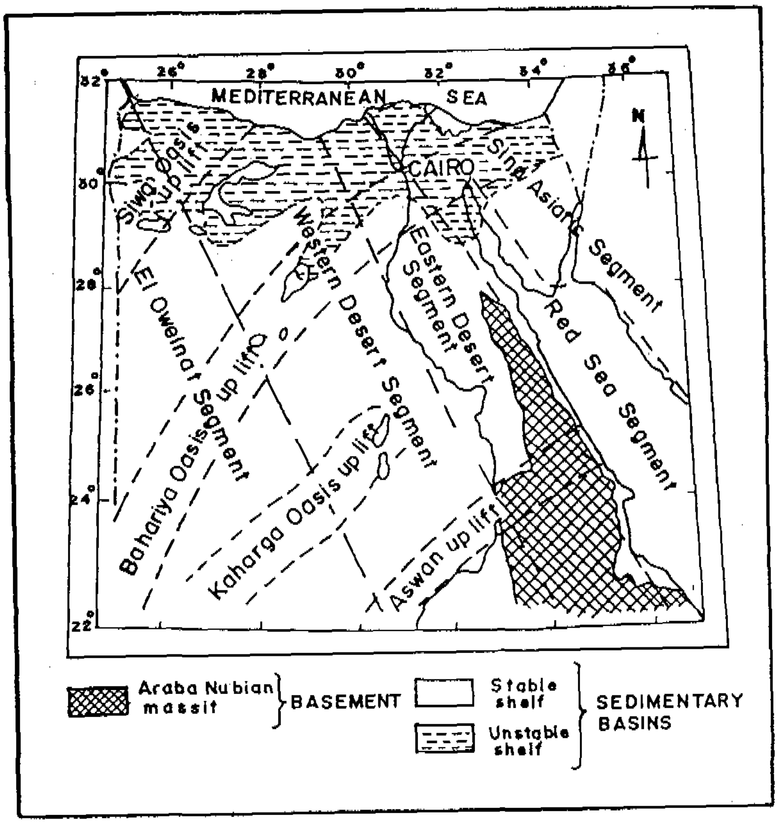

Figure 1. Tectonic Map of Egypt

3. GENERAL CHARACTERISTICS OF THE NILE AQUIFER SYSTEM

The Nile aquifer system extends from Aswan to the Mediterranean Sea. It can be divided into two regions, the Nile Valley (Aswan-Cairo), and the Nile Delta (below Cairo).

The Nile Valley and Delta regions (approximately 12,000 and 50,000 km^2, respectively) are morphologic depressions filled with Pliocene and Quaternary sediments (graded sand and gravel), mainly overlain with Holocene semi-pervious clay. The aquifer is generally confined from both sides and from the bottom by limestone formations (see Figure 2). The average width of the Nile Valley aquifer is about 12 km with an average thickness of about 100 m. The average width of the Nile Delta aquifer is about 200 km with an average thickness of about 200 m. The storage capacity of both aquifers is about 500 billion m^3. From the east and west, the aquifer is in direct contact with the Plio-Pleistocene aquifers underlying the adjacent desert fringes. On the fringes, huge reclamation schemes are taking place on surface water alone, on surface water and groundwater (old new lands), or on groundwater alone (most new lands).

The main source of recharge to the aquifer is the seepage from the irrigation distribution system and subsurface drainage. Discharge from the system takes place through groundwater flow to the Nile (especially in the region Assiut-Cairo), and groundwater extractions (amounting about 3.8 billion m^3/year at present, including the fringes with Nile water irrigation). Recharge from and discharge to adjacent aquifers are very limited. On the other hand, upward leakage in the Northern Delta, although minor, is of great impact on the salt-fresh water balance of the region. In general terms, the Nile aquifer system may be considered a closed system, except from the top.

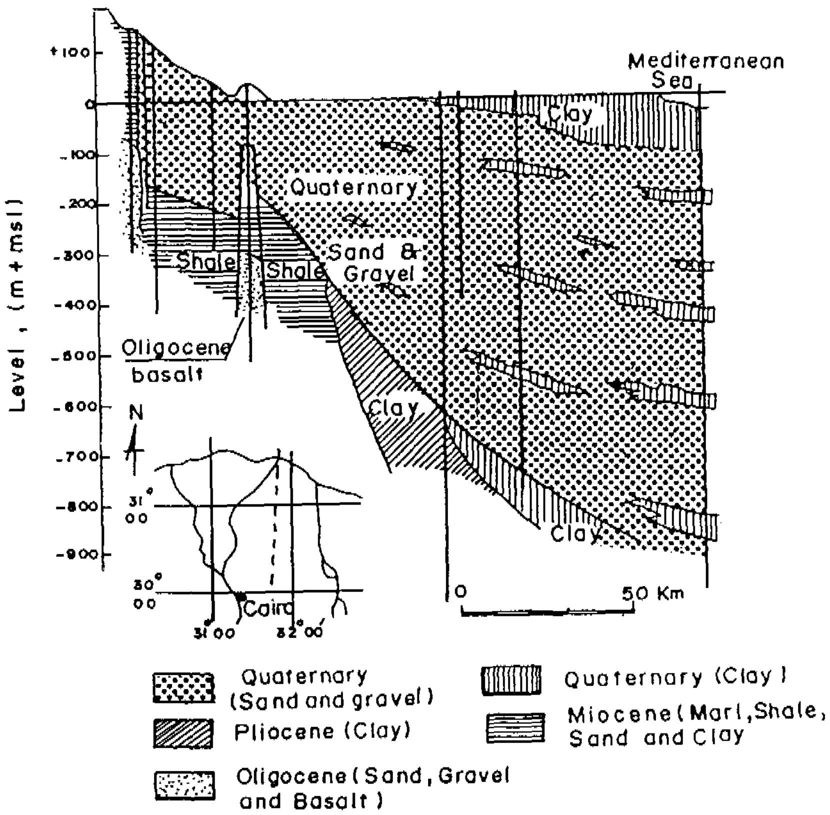

Figure 2. A Typical Lithological Cross-Section In the Nile Delta Aquifer System

4. GROUNDWATER DEVELOPMENT FOR DROUGHT

Satisfaction of water demands for drought conditions can be achieved by various means. Construction of surface water storage reservoirs and/or regulation of the aquifers as underground reservoirs may be considered solutions for drought. Surface reservoirs may be confronted by environmental and/or economic constraints that reduce their feasibility. Moreover, the conveyance of water from these reservoirs may be subjected to pollution and/or high water losses and delivery costs.

Aquifers can play an important role in such situations. Under certain conditions, aquifers can be considered potential seasonal and/or long-term storage reservoirs, along with serving as conveyance media. Groundwater storage can be one of the most efficient mechanisms for safe seasonal and long-term deficits in surface water. The storage capacity of a groundwater reservoir basin is analogous to the storage capacity of a surface reservoir, without the loss of water through evaporation which is a characteristic of surface reservoirs. Groundwater can be pumped locally, irrespective of the recharge locations. In addition, under certain conditions, groundwater withdrawals may result in drainage provisions (tube-well drainage), thus eliminating the need for extensive drainage networks and costly pumping stations.

4.1 Groundwater Development Strategies

Groundwater development, in general, can be achieved through a number of strategies: (i) continuous/all year round regulation; (ii) intra-annual regulation; and (iii) inter-annual regulation. Under each of these strategies, various development schemes could be thought of. Examples include satisfying domestic demands as first priority, conjunctive use of surface water and groundwater for irrigation (old or new lands), tube-well drainage, etc.

Drought periods are special cases of water shortages. They are generally characterized by wet and dry cycles that may extend for several successive years. For such conditions, the relevant regulation strategy of the groundwater system is the inter-annual. "Inter-annual" regulation involves removal of groundwater from storage to make up for shortages in surface water. This depletion of groundwater storage would continue for a period of a few years (depending on reverse conditions). Recovery of the aquifer would take place during periods of surface water surplus, in such a way that, in the long term, a dynamic equilibrium between groundwater extraction and recharge is maintained. Inter-annual regulation schemes could either stand for themselves (new constructions), or could evolve from annual or intra-annual schemes. This can be achieved by increasing the pumping hours or the pumping periods of the wells.

4.2 Constraints

Groundwater development may be affected by constraints that are either related to system limitations, or imposed by the developer. Major constraints to groundwater development include: (i) potential recharge; (ii) hydraulic characteristics; (iii) groundwater quality; (iv) boundary conditions; and (v) economic and socio-economic considerations.

The total amount of available groundwater depends on the potential recharge to the aquifer (annual and/or long-term). This includes both the deep percolation and groundwater flow from other aquifers.

The main hydraulic characteristics of the aquifer system that may affect groundwater development are the transmissivity (T) and storativity of the water bearing formations, and the hydraulic resistance (R) of the semi-confining layers (if any) to vertical flow. Low transmissivity values tend to delay the recovery of pumping from recharge areas to the well (groundwater flow through the aquifer). High values of R would delay the vertical replenishment of the aquifer (recharge).

The initial and long-term suitability of the groundwater to the allocated use should be considered in terms of individual ions. Induced polluted water may also be considered as a constraint, depending on the types of pollutants and the groundwater vulnerability.

Boundary conditions would affect the continuity of development strategies. Boundary conditions may include geological boundaries (no-flow), geological formations with low quality groundwater, or hydrologic boundaries (sea water).

The economic return from groundwater development is a very important issue to investigate. The main relative factors affecting the economy of groundwater development is the pumping head (static and dynamic heads) and the quality of the water (use). Under economic considerations all types of costs and returns should be included in the evaluation (including environmental impacts). Socio-economic conditions reflect the users preference for the use of a certain water source, and the impact of development on the existing schemes (hand pumps).

4.3 Groundwater Development in the Nile Aquifer System

Areas included are those free from any of the above mentioned constraints. Investigations concerning the environmental impacts, economic and socio-economic aspects should be carried out before final decisions are made.

Representative (sample) regions within the Valley and Delta are selected for the simulation and testing of various groundwater pumping schemes. East Delta is selected to represent Nile Delta conditions; while Beni Suef Governorate is selected to represent the Valley conditions (see Figure 3).

The package used for the simulation of the sample regions is "TRIWACO". TRIWACO is based on the finite element technique and can handle a variety of steady-state and transient groundwater flow problems.

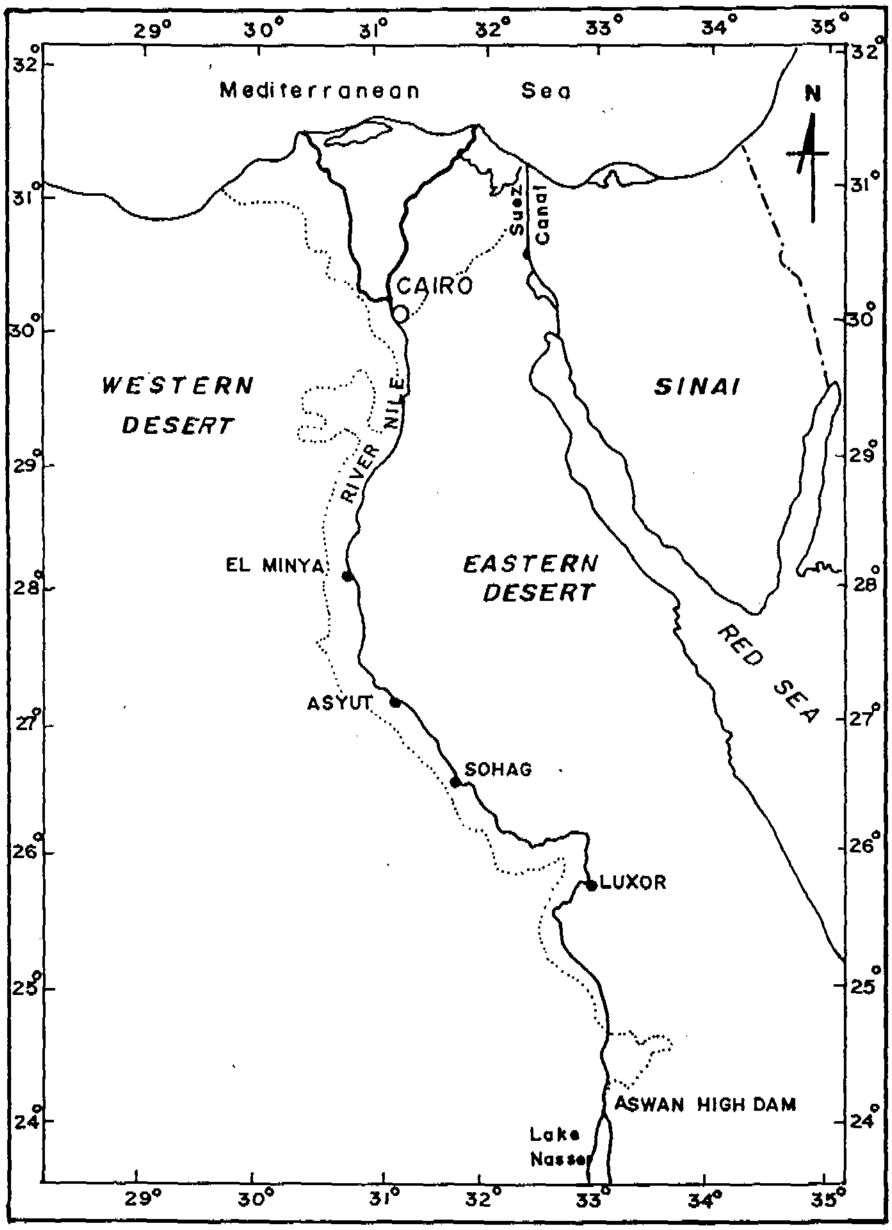

Figure 3. Location of Sample Areas

4.4 Predictions

Various distributed pumpage thicknesses are tested, starting from the minimum figure obtained from the calibration of the models. For each trial, various components are checked: (i) maximum regional drawdown (<3m); (ii) change in the fresh water thickness (<10% the original thickness); (iii) inland movement of the fresh-sea water interface (almost 0); and (iv) change in storage.

Figures 4 and 5 show the discretization of the simulated regions. In the Nile Valley, the distributed pumping thickness is 1.1 and 2.2 mm/day in the middle and fringes areas, respectively, with surface water reclamation schemes on the desert fringes. In regions with groundwater reclamation, a uniform pumping thickness of 1.1 mm/day is applied to both sub-areas. A strip one kilometer wide along the river course is left without pumping.

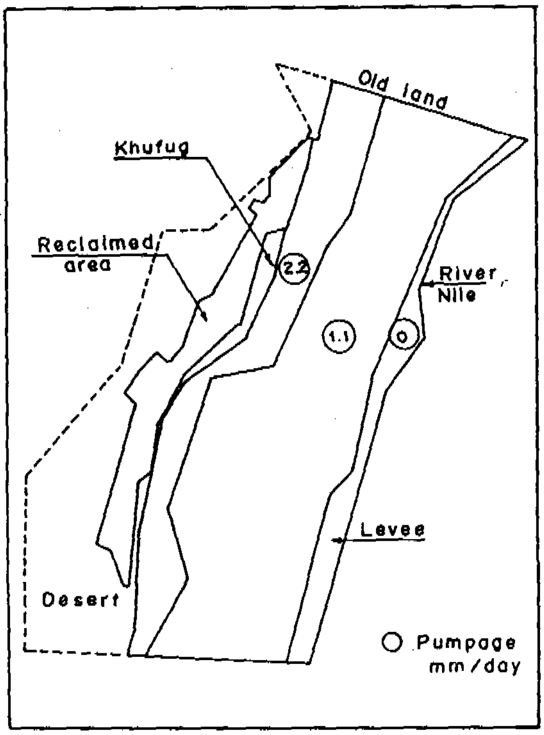

Figure 4. Boundaries and Distributed Pumpage In The Simulated Valley Region

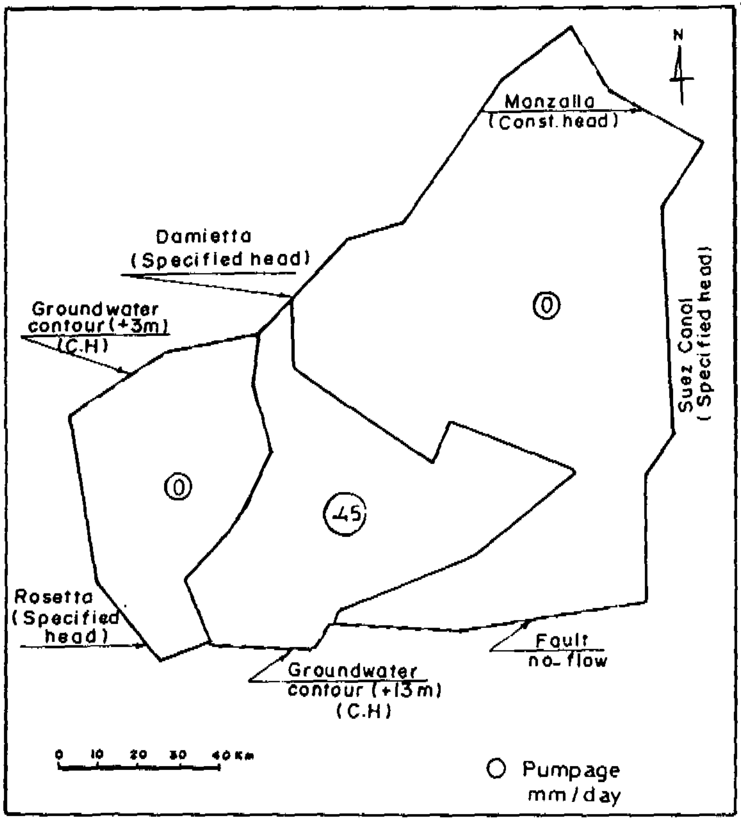

Figure 5. Boundaries and Distributed Pumpage In The Simulated Delta Region

In the Nile Delta, the distributed pumping thickness differs greatly from one subarea to another, depending on the fresh water thickness and location with respect to reclamation schemes (0 to 1 mm/day). A strip in the north is left free of pumping to prevent inland movement of the interface.

Tests are also made on the maximum possible period (successive years) of pumping that does not result in adverse impacts. This is found to be a maximum of 5 years, after which the system recovers during a period of 3 years with normal (present rate) pumping. Figures 6 and 7 show the cumulative change in storage in the Valley and Delta, respectively; while figure 8 shows the change in fresh water thickness due to sea-water intrusion.

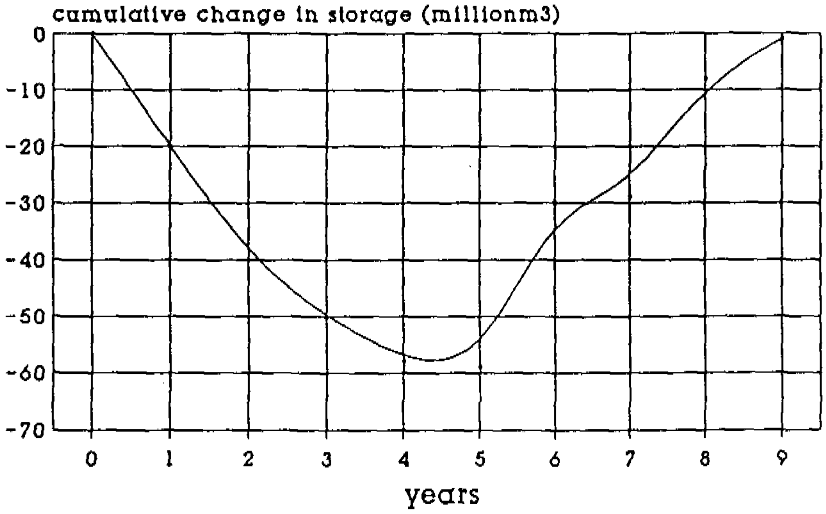

Figure 6. Cumulative Change in Storage For Inter-Annual Regulation In the Valley Region (million m^3)

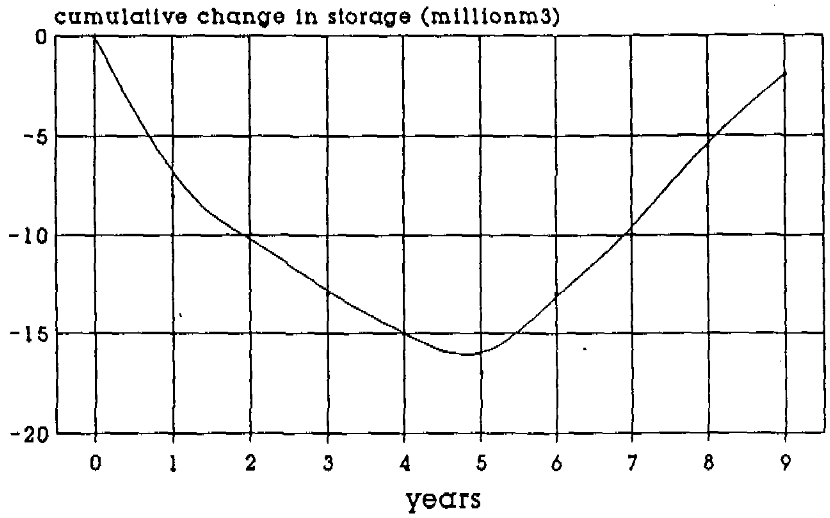

Figure 7. Cumulative Change in Storage For Inter-Annual Regulation In the Delta Region (million m^3)

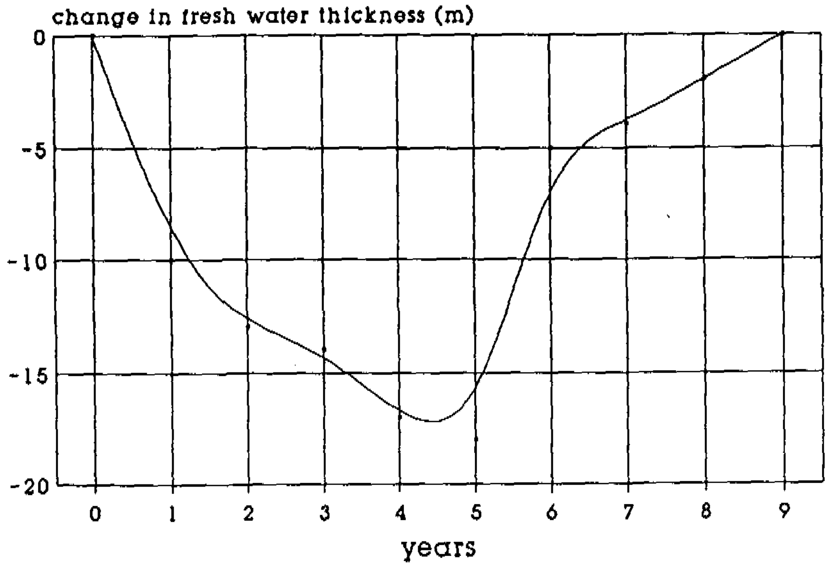

Figure 8. Change in Fresh Water Thickness

Results are extended to the other regions of the Delta and Valley, respectively, to determine the total amount of groundwater that could be used to make up for five successive drought years as follows:

1. In the Nile Valley, the area underlain by a promising aquifer is about 6,900 km^2; of which: (i) 900 km^2 are within the levee area (1 km strip along the river); (ii) 2,400 km^2 are in regions adjacent to surface water reclamation schemes (of which 480 km^2 are in the fringes, and 1,920 km^2 in the middle flood plain); and (iii) 3,600 km^2 are in regions with no surface water reclamation schemes (of which 720 km^2 are in the fringes area, and 2,880 km^2 in the middle flood plain).

2. The area of the Eastern Delta region subjected to groundwater development, as modelled in this exercise, amounts to about 3,611 km^2. In the Western and Middle Delta, the possible areas are 1,937 and 3,485 km^2, respectively.

3. Table 1 summarizes the results of the exercise.

4. The present rates of groundwater withdrawals amount to about 3.8 billion m^3/year. These can be doubled for five successive years during drought periods (in addition to groundwater pumpage in the groundwater irrigated fringes); then groundwater withdrawals set back to initial conditions for 3 years. Figures concerning groundwater potential for development in the groundwater irrigated desert fringes are estimated for normal long-term sustainability.

5. SUMMARY AND CONCLUSIONS

In Egypt, a variety of aquifer systems exist. Management and development of such aquifers is a must, especially for the cases of crisis (droughts). Groundwater in the Nile aquifer system is not a resource in itself as it is replenished from irrigation activities originating from the Nile surface water. The other aquifer systems include the Nubian sandstone and the Moghra, containing fossil water; the carbonates with brackish water; and the coastal aquifers with marginal quality water.

Table 1. Groundwater Potential in the Nile Aquifer system For Drought Conditions (million m^3/year)

locality	present extraction	drought extraction	TOTAL drought
NILE DELTA			
* East	731	630	1,361
* West	428	330	758
* Middle	652	610	1,262
* Land reclamation*	610	134	744
* Land reclamation**	611	86	697
TOTAL DELTA & FRINGES	3,032	1,790	4,822
NILE VALLEY			
* Flood Plain	1,155	2,000	3,155
* Land reclamation*	99	242	341
* Land reclamation**	151	358	509
TOTAL VALLEY & FRINGES	1,405	2,600	4,005
TOTAL NILE AQUIFER SYSTEM & FRINGES	4,437	4,390	8,827

* surface water irrigation; ** groundwater irrigation

The Nile aquifer system is almost the only renewable aquifer system in Egypt (with no recharging major works). The main source of recharge to the system is the seepage from the irrigation distribution system and subsurface drainage. Recharge from adjacent aquifers can be neglected compared to the total capacity of the aquifer. Discharge from the system takes place through groundwater flow to the Nile (especially in the region Assiut-Cairo), and groundwater extractions (amounting about 3.8 billion m^3/year at present). Discharge to adjacent aquifers and upward leakage in the Northern Delta are minor. Accordingly, the Nile aquifer system may be considered a closed system.

The Nile aquifer system is a huge storage reservoir analogous to the High Dam reservoir. Its total storage capacity is estimated at 500 billion m^3. Its hydraulic characteristics are suitable for the storage and transmission of water.

Development of the groundwater in the Nile aquifer system to mitigate drought effects can be achieved with almost no violation of the imposed constraints. The possible rate of groundwater withdrawals from the Nile aquifer system is estimated at about 4.4 billion m^3/year for five successive years, in addition to the present groundwater withdrawals, resulting in a total of 8.8 billion m^3/year. This is equivalent to about 16% of the present surface water deliveries from Aswan High Dam.

6. RECOMMENDATIONS

In this exercise, only the Nile aquifer system has been tested. In fact other aquifers could be developed for cases of crisis if properly approached.

The Nubian sandstone aquifer is a huge reservoir containing generally fossil groundwater of good quality. The contained groundwater can be regarded as a strategic reserve for cases of crisis, while normal development should be carried out carefully.

The coastal aquifer systems, although of small storage capacity, can be regulated and their capacity enhanced if proper water harvesting means are developed and applied. Such methods can make use of fresh Nile water disposed in the sea during the winter closure, rain water, and treated sewage water. Water harvesting and groundwater recharge are highly recommended in these regions.

Groundwater in the plio-pleistocene aquifer can be augmented by recharging the aquifer with sewage or drainage water. This aquifer may then be considered as a strategic reservoir for cases of crises.

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SATISFYING FUTURE WATER DEMANDS OF NORTHERN LIBYA

Saad A. Algariani[1]

ABSTRACT

Located within the hydroclimatic water stress zone, the northern regions of libya experience an ever widening gap between available water supplies and demand for water use. The resulting deficits have already imposed severe constraints to further economic development. Corrective measures include both the continuous search for further water supplies and the establishment of conservation-oriented water management. The first measure is monumentalized in the water transfer and redistribution scheme known as the Great Man-made River (GMR), with all its phases. The second measure is sought through the adoption of a demand water management system based on improved institutional capacities and the introduction of water pricing and banking systems, appropriate technologies and efficient water use in all sectors of economic social activities.

1. INTRODUCTION

Similar to several other countries in the Middle East and North Africa which are deprived of perennial rivers Libya is exposed to intensive hydroclimatic aridity which imposes severe constraints on its future sustainable development and welfare. The present annual per capita share of the potentially available water supplies does not exceed 500 cubic meters per person per year for all water uses. According to the concept of water stress index developed by Falkenmark (1990) this value puts the country deep into the water stress zone making it the poorest North African country in available water resources. In addition most of these available supplies are derived either from groundwater aquifers of non replenishable fossil water or pumped from rechargeable aquifers at a rate higher than their potential safe yield.

By virtue of their agricultural resources and relatively more favorable climate, the northern coastal plains, especially those in the northwest and northeast, are the most heavily populated and economically vital regions of the country. Up to the 1950's and early 1960's the human and economic activities in these regions had been stabilized in a delicate balance between available natural resources and sociopolitical relations based on subsistence rainfed agriculture and local forms of nomadism. This balance has been distorted lately by the ever increasing expansion and intensification of irrigated agriculture and urbanisation.

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The intensive mining of the local aquifers in these regions during the last decades has led to severe problems of groundwater pollution and water quality deterioration by sea water intrusions along the coastal areas. The resulting undesirable ecological and economic impacts of these problems have raised great concern among both public officials and private citizens (Alghariani, 1988). To ensure sustainability of the water resources development and the economic activities dependent on it, the present situation must be immediately ameliorated and the groundwater overdraft should be stopped or reversed. Hence the need for an integrated water master plan which provides for the present and future water needs of the country within the sustainabiliy imperative of economic and social development. The basic features of this plan and the major projects upon which it should be based before its final integration will be briefly discussed in the following sections of this paper.

2. POTENTIALLY AVAILABLE WATER RESOURCES

2.1. Surface Water Resources

Winter season rainfall is concentrated along the coastal strip and around the western and eastern mountainous areas where annual precipitation ranges from less than 100 millimeters in certain parts of the Golf of Sirte up to more than 500 millimeters in the Green mountain areas. Deeper into the southern deserts it tapers down to less than 5 millimeters. It is highly sporadic and variable in both space and time. The orographic and convective nature of the rainy storms are characterised by showers of high intensity and low duration which usually result in flash floods and significant amounts of run-off carried by ephmeral streams to be drained either into the Mediterranean sea or spread across the flat plains downstream of the wadis. During the last twenty years all these wadis have been dammed to conserve run-off water and prevent soil erosion.

In spite of the fact that more than 30 billion cubic meters of rain water falls annually on the northern parts of the country only less than 120 million cubic meters can be collected and stored behind dams at the present time. This low efficiency of run-off collection is due mainly to the unusually high infiltration rate of the predominantly sandy soils and the tremendous evaporative demand of the atmosphere. Rainwater harvesting practices have been introduced to increase rainfall efficiency in augmenting the available water supplies to provide drinking water for both humans and animals as well as to increase the productivity of rainfed agriculture and grazing lands. But all these practices, although highly important, are not expected to contribute significantly in meeting the overall present or future water demands of the country.

2.2. Groundwater Resources

Apart from the very limited amounts of runoff surface water mentioned above and negligible quantities of water obtained from nonconventional resources, the major total available water supplies are mainly derived from groundwater which is neither equally distributed within the country nor proportionate to demand for water use. Although several aquifers of low productivity and limited extent and storage capacity are present in the northern regions, the huge and extensive highly yielding groundwater basins of Kufra, Sarir and Murzuk are found in the southern parts of the country. Most of the population and its diverse economic activities, however, are concentrated in the northern parts of the country. above latitude 28 North (see Figure 1). Water demand in the southern parts, which are mainly desert and sparsely populated, is small in proportion to its relatively huge groundwater resources. Table (1) indicates the regional groundwater balance within the country during the year 1985.

There has been a continuous controversy and hot debates (Ahmad, 1982) about whether the sourthern aquifers are rechargeable or they are only made of fossil water which is continuously mined and does not receive any replenishment. This controversy had been fully discussed elsewhere (Ahmad,1993 and literature therein). The data presented in Table (1) is based on the assumption that the southern aquifers receive no recharge and the amounts of annual extractions result in a drawdown of one meter per year for 50 years of economic pumping.

It is clear from Table (1) that the northern regions already suffer a water deficit of 855 million cubic meters per year(MCM/Y) which is balanced by a severe overdraft mining of the local aquifers, especially in the northwestern part of the country. The negative effects of this overdraft been apparent since the early 1980'S (Alghariani, 1988) and are reflected at the present time in the continuous decline of groundwater piezometric levels and the inland advance of sea water intrusions along several parts of the coastal areas. If this overdraft is not stopped or reversed, it is expected that these intrusions will lead to the contamination and pollution of all the productive aquifers by the year 2000.

2.3. Nonconventional Water Supplies

There has been several attempts during the last twenty years to introduce and expand waste water treatment facilities and sea water desalination plants. By 1995, the targeted design capacities can reach up to 240 MCM/Y of treated wastewater to be used for agricultural production of forage crops and up to 100 MCM/Y of desalinated fresh water form the sea to meet part of the municipal requirement of the urban centers along the coastal areas, especially those threatened by severe sea water intrusions. This is unlikely to be achieved, however, unless the restrictions imposed by the high costs of energy and spare parts and the required skills for operation and maintenance are alleviated. Present levels of production are estimated at 30 MCM/Y of desalinated sea water and about 100 MCM/Y of treated wastewater. But in view of the increasing water scarcities from conventional sources and the rising opportunity cost of water in the various sectors of the economy, the nonconventional water resources offer a highly promising alternative, and they may even prove to be, on the long range, the only resort to meeting the future water demand of the country.

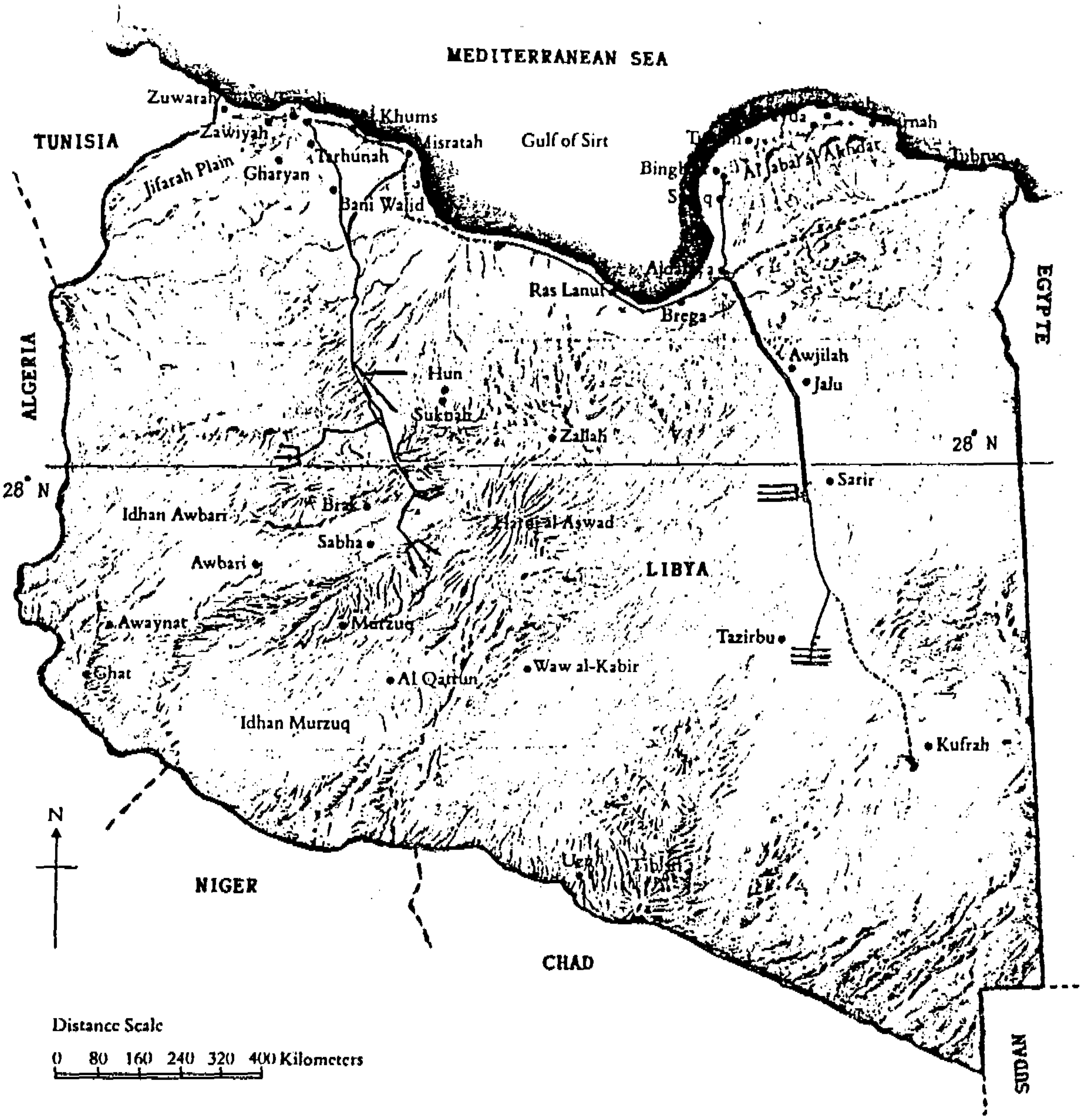

Figure 1. Physical map of Libya showing the regional water transfer and redistribution scheme (GMR)

Table:1. Within country groundwater balance estimated during 1985 (million cubic meters per year MCM/Y)

Region	Safe yield	Annual extraction	Water balance
Above 28°N	745	1600	-855
Below 28°N	4610*	750	+3860

* Believed to be a fossil water available on the basis of a drawdown of 1 meter per year for 50 years of economic pumping.

3. WATER TRANSFER AND REDISTRIBUTION

3.1. Project Conception

The regional past and future trends in potentially available water supplies from different sources and expected demand for different water uses are presented in Figure(2). It seemed during the late seventies and early eighties of this century that the only feasible way to stop the overdraft of the limited groundwater aquifers along the coast is to balance the available water resources budget. Possible savings through improved water use efficiency and better management, although highly desirable and urgently needed, are not sufficient alone in the face of escalating demands for water of a fast developing oil based economy geared to achieve a standard of living comparable to that of the industrialized western countries.

Inspired by the facts presented in Table (1), the answer to balancing the water resources budget was sought in transferring the extra groundwater supplies in the southern region to the water deficit areas of the northern region. Economic feasibility studies indicated that the cost of transferred water is much cheaper than any other alternative source and does not exceed 10% of the cost of sea water desalination. Thus the idea of the impressive network of the Great Man-made River (GMR) was conceived. Several issues, however, have to be clarified. These will be discussed below in more detail.

3.2. Strategy Considerations

The (GMR) water transfer and redistribution project fully recognizes the following principles which were taken into account during its formulation and implementation:

1. The present and future water requirements of the southern regions should be fully met and safeguarded. In other words, the transferred or exported water from the basins in these regions must represent a surplus after including all the needs of the local activities in the reasonable foreseable future.

2. The water requirements of the northern regions to which water is transferred or imported must be reduced to the minimum possible amount through tapping all alternative water resources which may be cheaper on the long range than the cost of the transferred water. In addition, all effective savings in existing uses should be implemented without impairing economic production efficiency.

3. It has not proven yet that the southern exporting aquifers are hydraulically connected with any recharge water sources in the area. Thus it is considered that the transferred water from these aquifers is not naturally replaced once it is withdrawn. Therefore its development has been undertaken with the full understanding that it will be depleted within a limited period of time depending on the storage volume which can be economically transferred. This period has been estimated to be 50 years. Within this period, the project should generate economic

returns sufficient to develop other water resources, such as desalination, to replace the transferred water supplies in the importing regions.

4. It is recognized that a water transfer project, once made, becomes essential to the welfare, if not the existence of the people it serves. Thus the project must be continued in service or substituted by another source of water. Otherwise, all the economic and social activities based on the project cannot be sustained in the future. The imported water will be a new element added to the physical environment of the northern region. The addition of this water certainly enhances the economic development and population growth in the region. If the transferred water were to be discontinued due to aquifer exhaustion or any other reason, the human activity in the northern region would experience catastrophic curtailment unless other alternative supplies are secured.

5. It has been realized that the project will have profound economic, hydrological and human environmental impacts, both during its construction period and throughout its operating life time. Whatever the economic, social and environmental costs which may result in the southern and northern regions, these costs must be born by the nation or regional authorities once the project is constructed and operating. Such costs should be mitigated when recognized in advance. They must be compensated for by water users through an efficient and effective water pricing system.

3.3. Legal Issues

The legal status of internationally shared water resources is still controversial and unclear especially when applied to groundwater resources. The non binding recommendations of the "Helsinky Rule" as formulated by the International Law Association (II.A) in 1966 give provisions and guidelines for cooperation among copartners in resolving issues related to shared international river basins. Although some international water law experts believe that the basic principles of the Helsinky Rule may be extended to shared groundwater resources, several countries and law authorities refuse this interpretation.

In the case of the southern Libyan aquifers the following facts can be stated:

1. Although some hydrogeologists believe that these aquifers are shared with the other neigh bouring countries of Libya, it has conclusively proven yet that these aquifers are continuous or hydraulically connected.

2. There is no evidence yet that these aquifers are partially or totally recharged from the Nile basin through the Nubian sandstone geological formations.

3. There has been no evidence from pumping tests and simulation models of operating well fields in these aquifers which indicates that a transboundary groundwater flow will be induced after the well fields designed to feed the GMR water transfer systems are operated.

Thus what can be said at the present level of available information is that:

1. The underground Kufra-Sarir and Murzuk basins are considered as non-flowing stagnant water bodies deprived of any proven horizontal physical mobility between sources and sinks, whereby dependent users along their route of motion may be quantitatively or qualitatively beneficially or harmfully influenced by other's uses.

2. Unlike running waters across natural river basins, these underground water basins should be considered as any other natural resources of vertical utility such as atmospheric resources, natural vegetation, soils and mineral deposits. As such, Libya, like any other sovereign country, can use this resource according to her present and future needs in the same way she uses the other land and atmospheric resources available to her within the boundaries of her territorial sovereignty .

3. Natural resource migration by inducement should neither be looked upon as a misuse of the resource in question nor a harmful action to copartners using this same resource. Otherwise, the world will be overwhelmed by geopolitical conflicts that may arise among cohabitant nations as a result of induced pollution, weather circulation, geomorphological changes and migratory herds, birds and humans.

Nevertheless, and despite all the above facts, Libya is willing to cooperate with all her neighbours in the scientific investigations and beneficial exploitation and use of the available groundwater resources close to their common borders.

4. THE (GMR) PROJECT

4.1. Project Phases and Stages

The Great Man-made River (GMR) project has been planned and implemented through five phases. The completion of these phases will provide the country with a comprehensive interconnected network of fresh water distribution which links together all major areas of water consumption and utilization. The total volume of water to be transferred and redistributed within the country through these five phases will amount to 2 billion cubic meters per year for at least a period of 50 years of economic pumping at the present levels of technology and market prices.

Phase one which has been completed recently, is expected to supply the north-central and north-eastern zones with a total of 700 MCM/Y which is believed sufficient to meet the near future water demands in these regions and to achieve the geopolitical objective of creating incentive to populate the long stretched and virtually empty space surrounding the Golf of Sirte. Phase two is currently under construction. It is intended to deliver 800 MCM/Y to the northwestern region, especially the Jafara plain and its coastal strip, in order to remedy the existing groundwater overdraft and to correct the environmentally harmful impacts associated with it. The other three phases are still in the preliminary design stage and their future implementation will depend on several factors which are unpredictable at the present time. The impact of the first two phases of the project on the overall projected future water demands of the country will be discussed in relation to Figure 2.

In relation to Figure 2 curve SS' represents the potentially available water resources in the northern region above 28 N° and curve DD' indicates the past, present and expected future demand for all water uses based upon the present rate of consumption, population growth and provisional plans for future economic development. It is clear from these trends that it is impossible to satisfy future demand with the expected water shortages without depleting the available groundwater supplies and contaminating the local aquifer reservoirs by sea-water intrusions.

Phase I of the GMR system was planned and executed during the early and late 1980s to transfer 700 million cubic meters per year (MCM/year) from the southern zone to the northern deficit areas at a continuous flow rate of 2 MCM per day, which will shift the supply curve SS' to the position SS'_1. This amount will become totally available to areas of utilization by the end of 1995 either at gradually increasing rate or all at once.

Phase II of the project is intended to bring 800 MCM of water per year at a pumping rate of 2.3 MCM per day to the northwestern part of the country by the end of the year 2000 to remedy the overdraft and meet future demands of this region. This will shift the supply curve SS' to the position SS'_2

4.2. Water Management Strategies

The water management issues involved in formulating acceptable policies and strategies of water utilization during this stage are very complicated. Difficult decisions have to be made with regard to economic, environmental and sociopolitical objectives of varying impacts and unpredictable consequences. There are several management choices, however. Among these:

1. The transferred water will be completely used to compensate for the overdraft and save the local aquifers from further deterioration. If this option is followed and demand water remain at it expected level represented by the curve DD', it is expected that the present shortages will be remedied by the year 2000 and there will be even excess water available for further expansion as indicated by the intersection of the supply curve SS', with the demand curve D.D'.

This is a highly desirable option from a water conservation and prevention of environmental degradation point of view. But since the transferred water will replace present water supplies which are invested free of charge in the economic activity of the region it is difficult, if not possible, to gain any return on investment or even to collect the overhead costs associated with the operation and maintenance of the system levying water utilization charges from users who are already accustomed to free water supplies. This option also robs

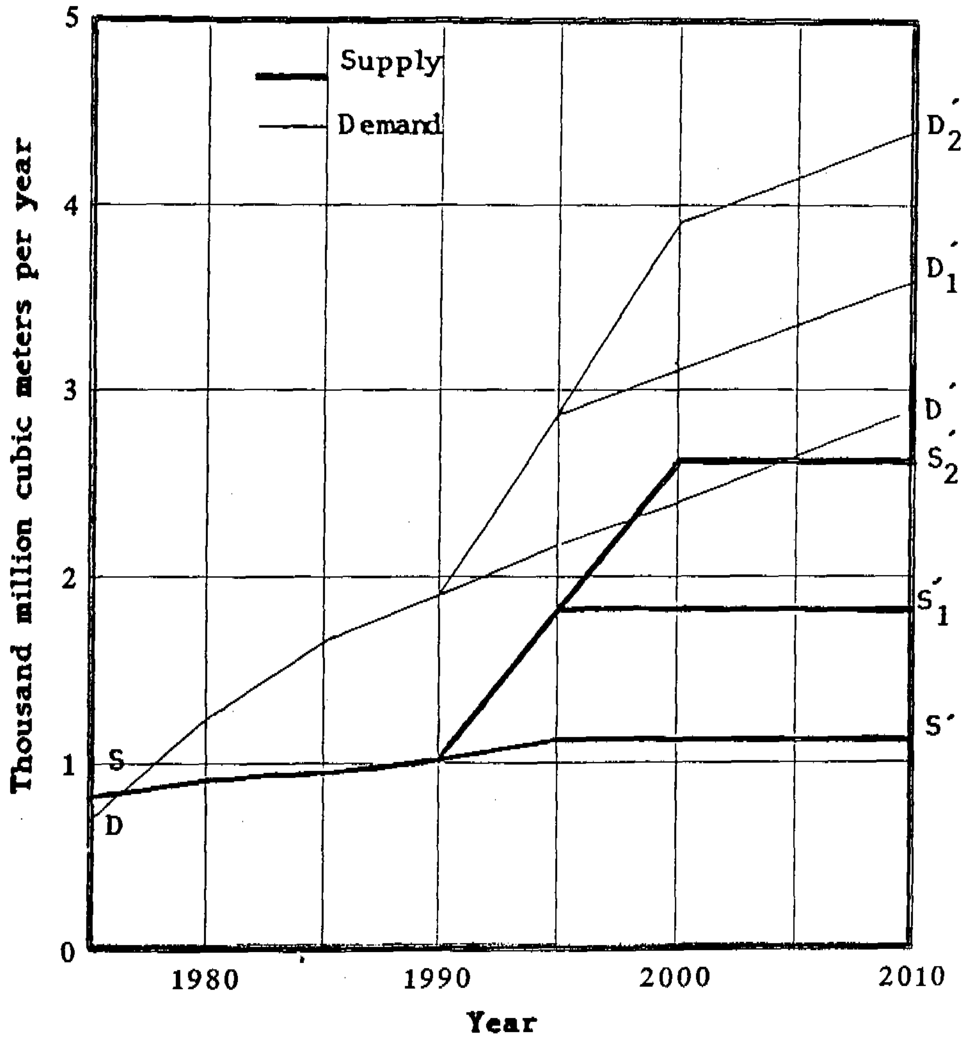

Figure 2. Trends in total water demand and potentially available water supplies.

the project of its impressionistic image and political influence which are better expressed through putting further land under irrigation and achieving economic equity by giving new opportunities to more people.

2. The second option is either to devote all transferred water of either phases or both of them to further expansion of irrigated agriculture and new industries and try to benefit from the newly generated income to cover the overhead costs of the project and even to recover part of its initial cost. This will shift the demand curve DD' to either DD'_1 or DD'_2, positions thus maintaining the initial overdraft and therefore does not result in any improvement of the present situation. In addition, this opinion provides employment to more people and helps in redistributing the national income as it does with the national supply of water resources. But the option overlooks the present water crisis in the other established sections of the economy and the dangerous future threats facing them. It also ignores the imperatives of sustainable growth and development which are the only secure measures for the continual survival and well-being of the nation and its future generations.

Needless to say that there are numerous other alternatives between the above two extreme options. These alternatives underwent several detailed analytical studies and critical evaluations to determine the best strategy for allocating the transferred water among different uses. As a matter of fact the decision has been already taken in favor of a more practical strategy which compromises the above two extreme choices by emphasizing sustainability and environmental safety while partially satisfying sociopolitical demands and economic returns.

4.3. Future Alternatives

The above discussions clearly indicate that the gap between water demand and total available water supplies as represented by the trends in Figure 2 will continue to widen and can never be bridged in the near foreseable future unless potentially new nonconventional water supplies are developed. Both groundwater and surface water supplies have been developed up to their full potential at the present time and even under the best management strategies they are not expected to add much to future water supplies without undesirable impacts. Thus the possibility to augment future water supplies in the face of an ever increasing water demand can only be achieved through expanding waste water treatment and reuse capacities to a full maximum potential which is relatively insufficient and/or turning to the seemingly inexhaustable resources of brackish and sea water by the adoption and development of desalination technology.

Another aspect of the available water resources has not been discussed openly in detail yet despite the fact that it may have far reaching local and regional sociopolitical consequences in the future. It was mentioned that the present water shortages will be alleviated by water transfer and redistribution through the GMR project with all its phases. But this project derives its water supply from the southern groundwater basins which are made of fossil water of no proven replenishment from any sources. If this is true then these basins, however huge their storage capacity may be at present time, will be exhausted and dried up sooner or later. The question which the country will have to face then is how to provide for the newly created communities and water consumption habits dependent on this project. The other important question to be answered is what will happen if the other neighbouring countries like Egypt, Sudan, Chad and Niger, under the pressure of population explosion and the need for economic development, start to develop the regions of these basins in their territories and compete with Libya for this nonrenewable water supply. In the absence of any formal agreement to regulate the allocation of this resource on an equitable and acceptable basis among these countries such a competition will certainly shorten the life span of economic exploitation of these basins. Unless other dependable water supplies are safely secured this situation may eventually lead to regional conflicts and political instabilities in the area. There are signs that such a drive towards competition may develop in the near future (Ahmad, 1993). Deprived from any other alternatives of conventional water resources Libya should prepare wisely for these future threats to its water security. Thanks to its oil wealth and geographical location, the country has both the natural and financial resources by which it can attract and encourage applied research and commercial construction of desalination facilities in order to develop and expand the technological base for this endevour.

5. DEMAND WATER MANAGEMENT

5.1. A Change of Emphasis

The fact that all renewable water resources have been fully developed and the southern aquifers feeding the GMR system will be eventually exhaused has driven the local water authorities during the last few years towards a continuous search for alternative water supplies, such as desalination and wastewater recycling to meet projected future water demands. While this search is essential for the future water security and development sustainability of the country, it is generally cost prohibitive at the present time and will not be expected by itself to solve future water problems unless supported by other sound measures of water control on the demand side of the national hydrological equation.

It has been realised recently that trends in future water demands should be always considered as future projections of the past and present misuses of the available water resources. As such, they reflect all the inherent inefficiencies of the present systems of water use in all sectors of the economy. These inefficiencies and wastes which amount up to 40% of the total water supplies can be significantly reduced through rational planning and management and the introduction of innovative and water-saving technologies. Some of the most relevant measures to be introduced and encouraged to achieve this objective will be briefly discussed in the remaining sections of this paper.

5.2. Water Pricing

It has been recognised by the local water authorities that subsidized water supplies priced lower than its opportunity cost is not conducive to efficient water allocations and uses among consumers. In addition to optimising water use, water pricing is also believed to generate extra revenue that can finance the continuous operation and maintenance of the water supply systems. But before the introduction of an efficient and attractive water pricing several sociopolitical issues must be resolved. First, the sociopolitical objectives such as food security, provision of drinking water and improving the income of rural people should be satisfied by other policy instruments. Second, legislative measures must be acted in order to create the local institutions needed to define water property rights and allocations among users as well as to determine water prices for different users and locations and to collect the accrued revenues.

5.3. The Introduction of Water Banking

The term "Water banking" is used here to describe the formal establishment of mechanisms and institutions which facilitate and promote the voluntary market exchanges and transfers of water between users (Miller 1993). These mechanisms and institutions must satisfy several requirements. First, water rights must be privately owned and clearly defined. Second, the benefits and costs associated with owning and using the water must accrue directly to the owner of water. Third, the water rights must be transferable by voluntary exchange (Tieteoberg 1988).

Based on purely technical reasons, there is a very promising potential success for introducing water banking to Libya. The water transfer scheme represented by the GMR project provides an ideal means for distribution of water between sellers and buyers of water. Since metering is essential for any successful water marketing scheme, the GMR system enables the transferred water to be routinely metered and reallocated much easier and less expensive than water obtained from other sources. In addition. this system also facilitates the easy movement of water between users who may be geographically distant from one another.

Libya's agriculture consumes 80% of the total available water supplies. Large volumes of this water are currently allocated below their coss of supply to produce low value crops. By shifting this water to higher value uses through water banking, both water and money are saved and the economy will develop on a more sustainable basis. It is also believed that if the Libyan farmers are faced with a higher prices for their water, they will be inclined to adopt and invent more efficient water use technologies. The water thus saved might then be marketed via the water bank. The farmers can sell a portion of their water allocations and use the proceeds to invest in more efficient water use technology.

5.4. Increasing Water-Use Efficiency

It seems clear that the heavy demands made by irrigated agriculture on the limited available water resources necessitate that the highest priority must be given to maximizing the amount of crop yield produced per unit of water supplied. This objective can be achieved through the introduction and adoption of water-saving technologies, improving system performance, realization of potential yields through increasing agricultural inputs of improved quality and by following management practices in water allocation based on comparative production advantages and opportunity cost of water.

It is possible to reduce irrigated areas and reallocate the saved water for utilization in other activities of higher economic returns. The increased income can then be used to import food at lower cost than if it is locally produced. The concept of water-use efficiency should be considered and applied in its broader sense which encompasses all technical innovations and management decisions directed towards maximizing the net benefit of each unit of water used. Intensive research is required to set priorities and formulate managerial policies concerning this issue.

6. SUMMARY AND CONCLUSIONS

The previous discussions in this paper have clearly indicated that Libya, with its fast rate of economic development and limited water resources, faces a serious threat to its welfare in the near future. This is reflected in the ever widening gap between increasing water demands and diminishing water supplies. It has been pointed out that within-country water transfer and redistribution projects, though might be considered as an expedient remedy, will not be able alone to meet the future water demands of the country. Besides the fact of being dependent on fossil water of non renewable nature, there is always the potential threat of increasing competition for this limited resource among the coriparian neighbouring countries which may eventually reduce the life span of economic utility of the water transfer projects. The only alternative left to the country is to devote its efforts towards developing other nonconventional water supplies of more dependable nature. Desalination of brackish and sea water offers a highly viable option. In view of the ample resources of fossil, solar and nuclear energy resources of the country and the promising innovations in desalination technologies, the possibility of obtaining desalinated water at competitive costs may be realised in the near future. Besides desalination, other alternatives include wastewater treatment and more efficient water use through better management. It is recommended that:

1. The country should embark on experimenting with the diverse innovative desalination technologies utilizing the various renewable energy sources available within its boundaries. The objective is to lay the foundation and local skills required for the extensive reliance on this industry in the future.

2. It is desirable to start negotiating acceptable agreements with the neighboring countries for the most beneficial uses of the shared aquifers and their management in such a way as to avoid any negative impact and sustain their productivity for the longest possible period of time. Detailed field investigations are required to further assess the extent and storage capacities of these shared aquifers. This may require the mutual pooling together of the experiences and know how of the countries concerned and probably some help from the international organizations and financial institutions.

3. On the demand side, the country must seek through all means available to reduce the demand for water and to increase water-use efficiency in all sectors of the economic and social activities. Most important in this respect is to reduce the amount of water wasted in irrigated

agriculture and reallocate the available water supplies on sounder basis of economic efficiency, according to the principles of comparative advantages and opportunity costs of water.

4. It is believed that water pricing and water banking can be introduced and implemented for the advantage of more efficient water allocation than the present practices. Besides providing more income to cover running costs of present water project they may lead to further savings in water use practices among consumers.

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THE STRATEGIC MEASURES FOR SATISFYING WATER DEMAND IN FUTURE

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ABSTRACT

During the past few years there has been an increasing importance of water in many countries, especially in developing countries. More and more countries will be facing the water crisis in the next century.

The annual amount of water resources in China takes the sixth place in the world in descending order, but it can be considered as abundant. The per capita amount of water resources is only about 1/4 of the world mean and the water amount per unit area of farmland is about 80% of the world mean.

The water demand in China increases rapidly due to development of national economy and improvement of human life. The total water demand at the end of this century will increase about 7 times in comparison with that at the middle of this century.

The possible strategic measures for satisfying the water demand in future could be divided into two categories: to save water and to use water as fully as possible.

In the first group of measures, various water saving measures for irrigation as well as those in industry and for domestic purpose are involved.

In the second group measures for storage of more water for further use, measures for interbasin water transfer, measures for utilizing all kind of water including reuse of drainage water and waste water, and measures for keeping the water quality in situation of suitable for use are involved.

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RÉSUMÉ

Pendant ces dernières années il y a eu une croissante importance de l'eau dans de nombreux pays particulièrement dans des pays en voie de développement, parmi eux des pays qui sont menacés par la crise de l'eau seront de plus enplus nombreux.

La quantité annuelle de ressource en eau en Chine prends sixième place dans le monde par ordre decroissante, mais il ne peut pas être considéré comme abondante. La quantité capitale de ressource en eau est seulement environ 1/4 de quantité moyenne mondiale et la quantité de l'eau par suface unitaire des terres cultivées est près de 80% de quantité mondiale.

La demande en eau en Chine augmente rapidement due à développement de l'économie nationale et à amélioration de la vie humaine. La demande totale finale en eau du pays sera augmentée de 7 fois environ par comparaison à celle du milieu de ce siècle.

Les mesures possibles stratégiques pour satisfaire la demande en eau à l'avenir ont pu être divisées en deux catégories: garder de l'eau et utiliser de l'eau le plus complètement possible.

Dans le premier groupe sont impliquées de différentes mesures de l'économie de l'eau pour l'irrgation aussi bien que des mesures pour l'industrie et domestique utilisation de population.

Dans le deuxième groupe sont impliquées des mesures de l'emmagasinage de l'eau pour en utiliser plus loin, celles de transfert de l'eau pour des inter-bassins, de toutes les espèces de l'eau et réutilisation de l'eau de drainage et perdue et des mesures à maintenir la qualité de l'eau dans la situation convenablement utilisée.

1. INTRODUCTION

During the past few years there has been an increasing importance of water in many countries, especially in developing countries. More and more countries will be facing the water crisis in the next century. The main reason for the mentioned situation could be listed as follows: the limited amount of available fresh water, the increasing growth of population, the increasing water demand in agriculture as well as in industry and for domestic purpose due to development of national economy and improvement of human life, the reduction of available water amount due to degradation of water quality. Most countries with frequently serious water shortage are those from Africa, Near East. At the same time over mining of groundwater has significantly occurred in some regions of China, India, Indonesia, Thailand, Mexico and some western parts of USA. Table 1 shows a brief statistical information data of per capita amount of water resources in different parts of the world.

Table 1. The per capita amount of water resources in different regions (in $10^3 m^3$)

Regions	1950 yr.	1960 yr.	1970 yr.	1980 yr.	2000 yr.
Africa	20.6	16.5	12.7	9.4	5.1
Asia	9.6	7.9	6.1	5.1	3.3
South America	105.0	80.5	61.7	48.8	28.3
Europe	5.9	5.4	4.9	4.4	4.1
North America	37.2	30.2	25.2	21.3	17.5

The annual mean runoff in China is about 2711.5 billion m^3, which takes the sixth place in the world in descending order. The large amount of water resources incorporated with other conditions has supported the development of such a large country for several thousands of years. At the end of 1980's China brings up about 22% of the total population by 7.2% of the total cultivated land in the world. But China is being short of water and land resources, because the per capita amount of water resources is only 2474 m^3, which is approximately 26% of the world mean. The available water amount per unit area of cultivated land is only about 80% of the world mean. The comparison of the water resources in some countries is shown in Table 2.

Table 2. Comparison of water resources in some countries

Countries	Annual Runoff (10^8 m^3)	Water Yield ($10^4 m^3/Km^2$)	Per Capita Water Amount (m^3/person)	Water Amount Per Unit Area (m^3/mu)*
Brazil	51912	60.9	42200	10701
Canada	31220	31.3	130080	4771
USA	29702	31.7	13500	1046
China	27115	28.4	2474	1888
India	17800	51.4	2625	721
Japan	5470	147.0	4716	8462
Whole World	468000	31.4	9360	2353

* mu: unit area of cultivated land, equivalent to 667 m^2

Due to the development of national economy and the improvement of human life, the annual water demand in China during the second half of this century increased and increases rapidly. The total water demand at the end of this century will increase about 7 times in comparison with that at the middle of this century. The growth of the water demand in China in shown in Table 3.

Table 3. The growth of water demand in China (in $10^9 m^3$)

Demand	1949 yr.	1957 yr.	1965 yr.	1980 yr.	2000 yr.
Agricultural	1001	1938	2545	3912	5100-5250
Industrial	24	96	181	457	1200-1300
Domestic	6	14	18	68	200-220
Total	1031	2048	2744	4417	6500-7000

It is obvious there will be serious contradiction between the water demand and the water available. A large amount of works has been done by the researchers, engineers and experts in various fields with the purpose of meeting the water demand in future. The possible strategic measures for satisfying the water demand in future could be divided into two basic categories: to use the water resources as fully as possible and to save the water as more as possible. As an old Chinese idiom says: to broaden the source of income and to reduce expenditure.

2. MEASURES FOR WATER SAVING

Strategic measures for reducing water expenditure are basically related to water saving management. Water saving should be covering all of the water user departements inlcuding agriculture, industry as well as domesticity. Recently, establishing a water saving type society is of increasing importance in China.

Agriculture is the largest water user in the national economy development. In recent years, agricultural water demand reaches approximately 88% of the total water demand, most part of which is due to irrigating farmland, but the water use efficiency for irrigation is about only 30-40% average in the whole country. Thus there are considerable potentials of water saving for irrigation. Large amount waste of irrigation water is being mainly due to losses of water seepage from canals and over irrigation leading to deep percolation. So in the essential measures of water saving for irrigation are involved the canal lining for reducing the water losses due to canal seepage and the application of appropriate irrigation techniques instead of using irrigation methods leading to waste of irrigation water in fields.

Canal lining has been developed considerably rapidly during last two decades. For over 50% of the main canals and 30% of the branch canals of irrigation systems in whole country have been lined, among which most canals located in arid and semiarid regions were lined completely.

For canal lining a wide variety of materials and techniques has been applied in practice, such as soil lime concrete, soil cement, stone masonry concrete, plastic membrane bituminous concrete, bituminous glass fabric sheet and some others. The canal lining could decrease the water losses due to canal seepage on 40-95%, depending upon the materials used. Effects of canal lining are shown in Table 4.

Table 4. Effects of canal lining on seepage control

Canal Lining Materials Used	Reduce of Water Seepage from Canal
Concrete of various types	85-95%
Stone masonry	40-60% , 80-90%
Soil cement	85-90%
Clays	60-80%
Bituminous materials	90-95%
Plastic membrance	90-95%

The application of low pressure pipeline network instead of open channels has been popularized since 1970's in the northern and northeastern parts of China. In some regions buried low pressure pipelines and in other regions soft pipelines were used. The buried pipelines are mainly made of local materials and cement, and the soft pipelines are made of plastic materials. Low pressure pipeline could save irrigation water by 20-30%.

As for irrigation techniques for paddy fields, a dry-wet-dry irrigation scheduling instead of basin irrigation has been applied increasingly day by day. In other cases sprinkler and drip irrigation have been used for appropriate crops such as vegetables, fruit trees and even dry-land crops. It could lead to save water by 40-50%. In some regions farmers use plastic film to cover the irrigated ground surface and it results in water saving by more than 50%.

There are also great potentials for water saving in industry. Recently the water reuse in industry is only 30-40% of the total water demand. This is a low level of water reuse for industry in comparison with the developed countries. It is expected to increase the water reuse for industry by 10-20% average at the end of this century.

Water demand for domestic purpose will be increased due to improvement of human life. It is expected to increase the per capita annual water demand to 110 m^3 in cities and 80 m^3 in rural regions for domestic use. At the same time it is also necessary to pay more attention to the water saving.

3. MEASURES FOR FULL UTILIZATION OF WATER

The total water resources in China is about 2800 billion m^3, including about 2700 billion m^3 of surface water resources. At the end of 1980's the total water use is about 450 billion m^3, in which the water use is only 16% of the water resources in the whole country and has been developed inequitably in different regions. Consequently, it is quite important to use the water resources as full as possible.

3.1 To Store Water As Much As Possible

Water resources in China are unevenly distributed in terms of time. The four consecutive months with plenty of runoff are March-June, April-July, May-August or June-September depending upon the local conditions. The total amount of water within these four months is about 50-80% of the normal annual value. In some regions most part of annual runoff is concentrated in two months, even in a few storms.

Due to the above mentioned reasons, construction of storage facilities for regulating the uneven distribution of runoff is being one of the effective measures for full use of water resources.

In the past 40 years since 1949, 86881 reservoirs of various sizes have been built, among which 328 are large reservoirs with storage capacity $\geq 100\times10^6$ m^3, 2333 are medium-sized reservoirs with storage capacity of 10×10^6 - $100 \times 10^6 m^3$ and 84220 are small ones with storage capacity of 10^5 - 10×10^6 m^3. A number of new reservoirs will be constructed and under construction in mountainous regions of South China, in upper and middle reaches of Yellow River, Huai River Basin and in North and Northeastern China. Three Gorges Project in Yangtze River and Xiaolangdi Reservoir in middle reach of Yellow River are being the huge ones under construction recently.

3.2 To Implement Inter-Basin Water Transfer

Water resources in China varies considerably from east to west and from south to north. However, the distribution of cultivated land is being quite different. The northern part of China has 64% of the total cultivated land but only 18% of the total amount of water resources, while the southern part has 82% of the water resources but only 36% of the farmland. A similar situation exists in some of smaller local regions as well. So there exists nonconformity between water and land resources. The characteristics of water resources in some river basins are shown in Table 5.

Table 5. Characteristics of Water Resources in Some River Basins

River Basins	Yangtze River	Huang-Huai-Hai Rivers	Hai-Luan Rivers
Basin area (10^3 km^3)	1800	1340	320
Annual runoff (10^8 m^3)	9510	1570	288
Per capita available water (m^3)	2500	457	262
Available water per unit area of farmland (m^3)	2700	360	170

Obviously, Yangtze River has 6-7 times of water resources, 5-6 times of per capita water amount and 9-10 times of available water per unit area of farmland in comparison to those of Huang-Huai-Hai Rivers. In these situations it is necessary to construct the water resources systems diverting the water from regions with abundant water to regions with less water in order to fully use water resources for more or less equitable economy development of different regions. In Northeasternn China, North China Plain, Yellow River Basin, Yangtze River Basin etc. there is a number of inter-basin water transfer projects, among which Projects diverting water from Yangtze River Basin to the north are excellent examples in China. It is so called south-to-North Water Transfer Project.

After investigation and analysis for many years three projects of South-to-North Water Transfer have been decided to be implemented step by step in future and some works of them have started recently. They are so called the West, the Middle and the East Route of South-to-North Water Transfer Projects in China.

These three projects are adapted to the three terraces of the China's continent. The West Route passes through the first terrace, the Qinghai-Tibet Plateau, or the east side of the Plateau. The Middle route passes through the east edge of the second terrace. The East Route is positioned on the third terrace on the coastal plain.

3.2.1 The West Route Project

The West Route Project has its purpose to transfer water from the upper reaches of the Yangtze River and its tributaries to the upper reach of the Yellow River, as shown in Figure 1.

Water is expected to be transferred from the Tongtianhe River, the Yalongjiang River and the upper reach of the Daduhe River, i.e. several tributaries in upper reaches of Yangtze River, to the upper reaches of the Yellow River.

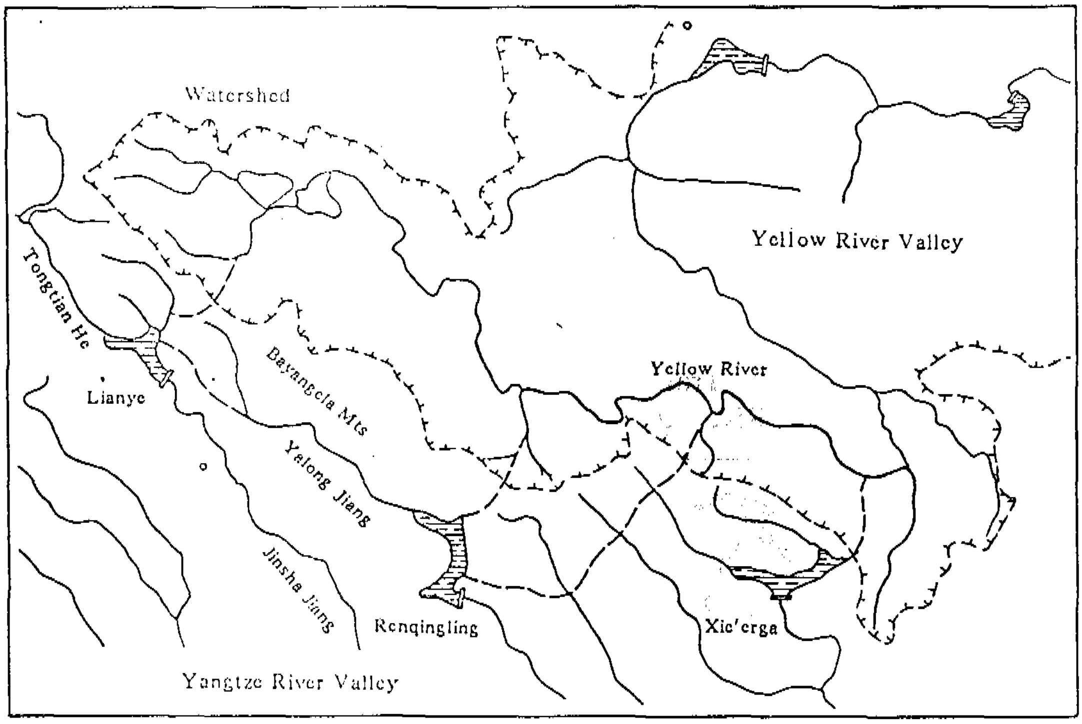

Figure 1. Sketch map of the West Route of South-to-North Water Transfer

Twenty billion m^3 of water are proposed to be transferred and merely 10-17% of the flow rate of the three tributaries of the Yangtze River. It is expected to increase the discharge of the Yellow River by one third. This is a strategic measure to make up for the deficiency of water resources of the Yellow River and to solve the problem of aridity and water shortage in the Northwest and North China. The gain in power generation on the upper Yellow River will also be expected and will be more than enough to compensate for the loss on the Yangtze River. Also, it would make the sediment control better in the Yellow River by improving the condition of water-silt proportion.

The construction site will be in the southeastern part of the Qinghai-Tibet Plateau, some 3500-5000 m above sea level. River stages at various section on the upper Yangtze River envisaged for diversion are 350-520 m below the elevation at the spot where the diversion channel is to empty itself into the Yellow River, and 500-600 m below the elevation of watershed between the Yangtze River and the Yellow River. High dams will be necessitated, together with need for long tunnels. The height of dams will be reduced if pumping is resorted to, effecting also shortening of the tunnels. Three dam sites, i.e. Lianye, Renqingling and Xie'erge, and some pumping station are under study, and tunnel dug through Mount Bayangela were planed. The proposal will involve huge volume of work and high cost construction.

3.2.2 <u>The Middle Route Project</u>

The Middle Route Project will transfer water from the middle reach of the Yangtze River and also its main tributary, the Hanjiang River, to the middle reach of the Yellow River. It may deliver water to most parts of the North China Plain by gravity.

The Middle Route Project will deliver the water northward from the Danjiangkou Reservoir constructed on the Hanjiang River. The water will be diverted from the Three Gorges Reservoir on the Yangtze River to satisfy the further demand of the water-shortage regions.

The Middle Route Water Transfer Project connects with four large basins, the Yangtze River, Huaihe River, Yellow River and the Haihe River, as shown in Figure 2.

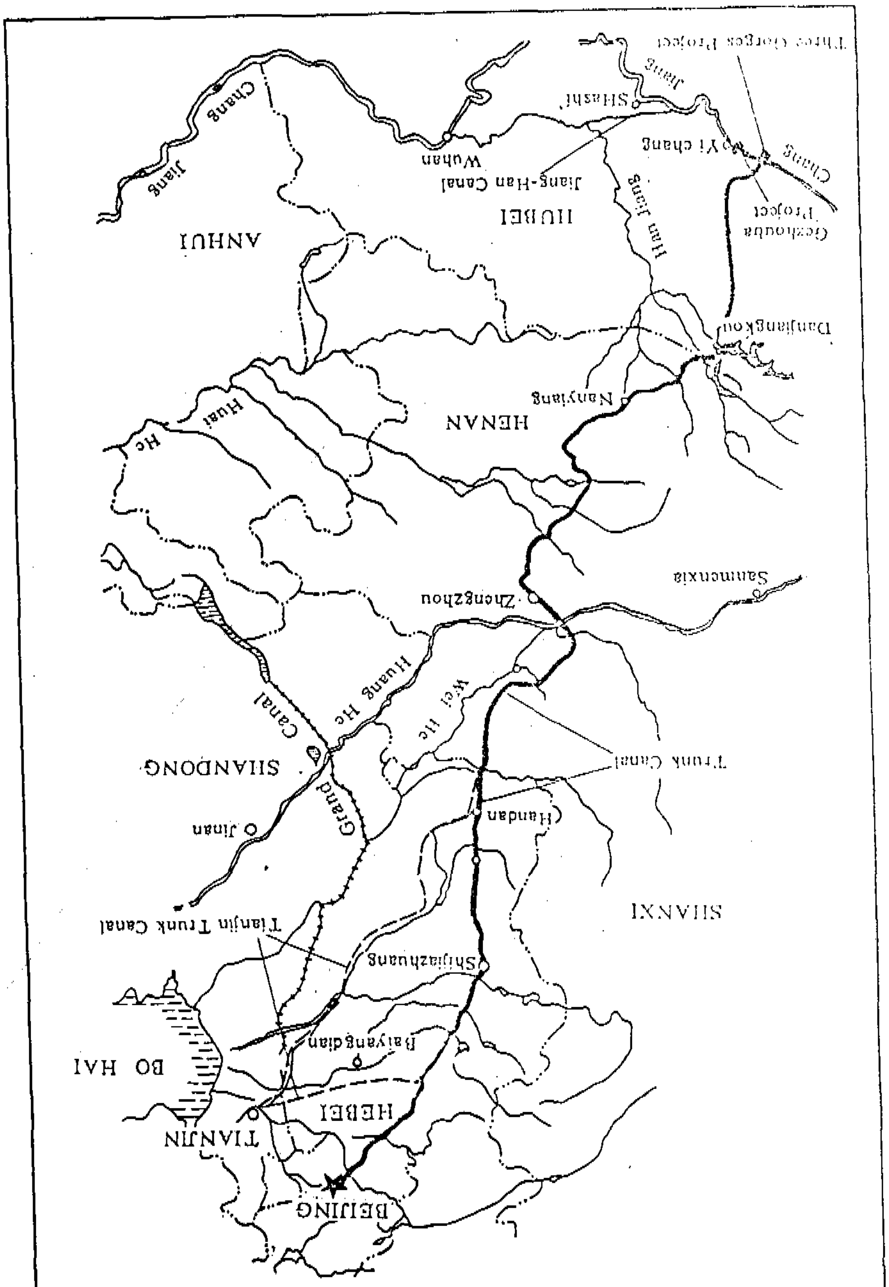

Figure 2. Sketch map of the Middle Route of South-to-North Water Transfer

The annual amount of water transferred by the project is 14 billion m^3. It will satisfy preferentially the municipal and industrial use of main cities, and also provide appropriately the agricultural water requirement to the cultivated lands along the trunk canal.

There will be no difficulty in technology that cannot be conquered.

The Trunk Canal from the head work to Beijing is about 1240 km long, with a natural drop of about 90 m providing excellent topographical condition for gravity water supply. The Tianjin Trunk Canal has a length of 142 km. About 95% of the Trunk Canal will be built in earth area with the ordinary hydraulic structures. On the Trunk Canal there will be a number of structures crossing through 169 large and small rivers.

But the Hanjiang River is being insufficient to satisfy the further demand of the water shortage regions, more water will be diverted from three Gorges Project on Yangtze River as the second stage of the Middle Route Water Transfer Project, which involves the expanding reconstruction of the Danjiangkou Project with increase of Normal Pool Level from 157 m to 170 m.

3.2.3 The East Route Project

The East Route Project transfers water from the lower reach of the Yangtze River with most abundant water resources. Along the ancient Grand Canal it supplements water to the east part of the North China Plain. Besides, it also restores and enlarges the tranport capability of the ancient Grand Canal. The sketch map of the East Route is shown in Figure 3.

The Project has many obvious advantages such as: it can make full use of the existing river channels, lakes and some completed projects; it may be constructed in stages in order to play the role of the comprehensive benefits of water supply, navigation and waterlogging control; it will be a project with large benefit and without any risks.

The first stage of the project will pump 600 m^3/s of water from the Yangtze River to north (8.85 billion m^3 per annum) and 200 m^3/s of water will be crossed under the Yellow River northward to Tianjin and Beijing in order to solve the problem of the urgent water demands for the development of industry, agriculture and navigation.

The final scale of the Project will pump 1000 m^3/s from Yangtze River to north and 400 m^3/s of water will pass through Yellow River to North China Plain. The total length of the East Route Water Transfer Project is about 1150 km from Yangzhou in Jiansu Province to Beijing. Water will be delivered from Yangtze River by means of existing Jiandu Pumping Station to the Grand Canal, and then will flow into the Hongze Lake, Luoma Lake, Nansi Lakes and to the south bank of the Yellow River. Along the route from north bank of the Yangtze River to south bank of the Yellow River 13 pumping stages will be established, including 22 pumping stations with the total lift head of 65 m to overcome the surface elevation difference between Yangtze and Yellow River.

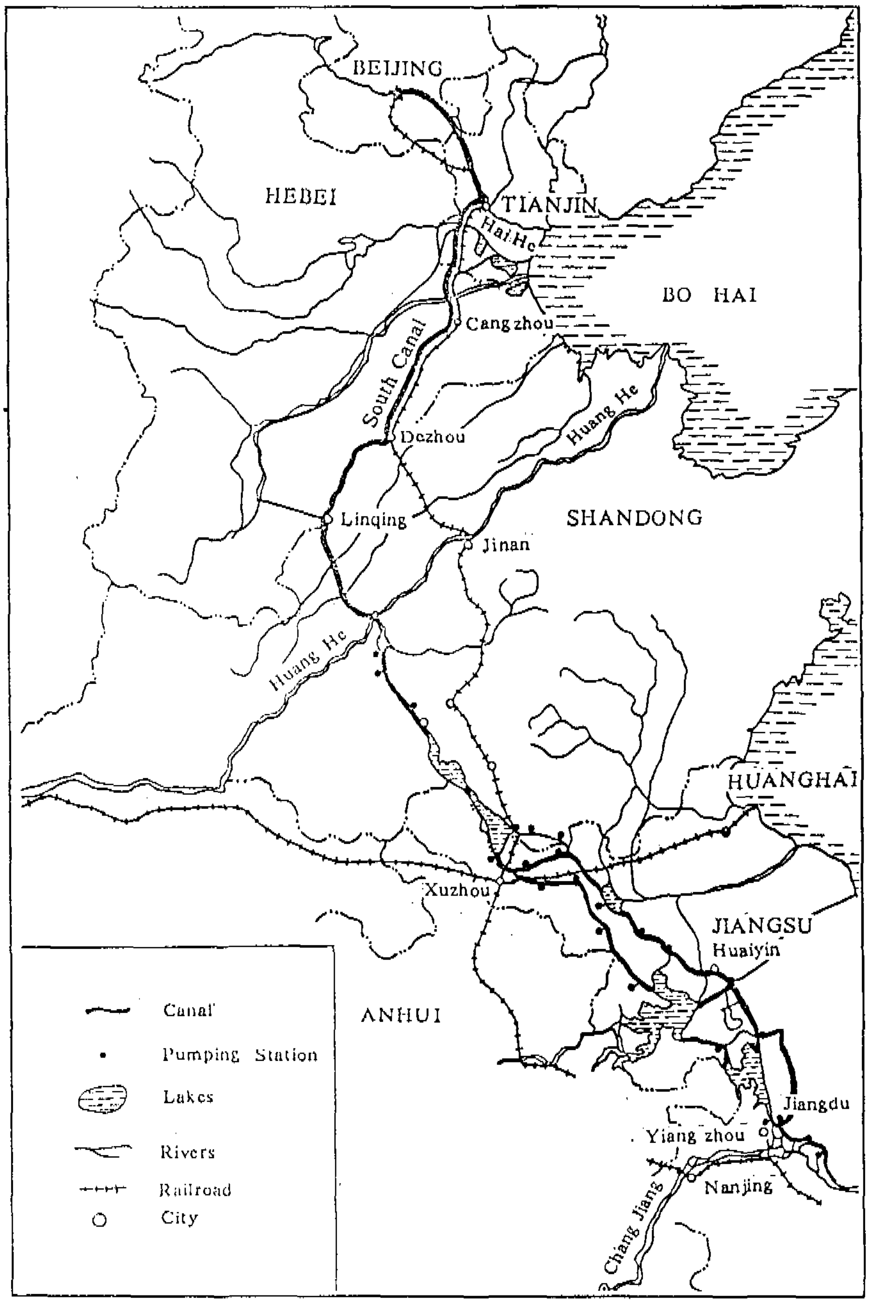

Figure 3. Sketch map of the East Route of South-to-North Water Transfer

Water transferred will be crossed under Yellow River by a three-pipe inverted siphon with diameter of 9.3 m each and length of 584 m.

After crossing the Yellow River, water will flow to Tianjing by gravity, and then to Beijing by pumping up 40 m.

The main problems concerned include: the impacts of water transfer on the lower reach and the estuary of the Yangtze River; whether or not the schistosomiasis will spread from the south to the north with the transferring water; whether or not the soil salinization in the North China Plain will be aggravated; whether the water quality is guaranteed or not etc.

After implementation of water transfer by the above mentioned three routes, Huang-Huai-Hai River Basins will be supplemented by 60-70 billion m^3 of water, i.e. and equivalent to 40-50% of the total runoff of these rivers, while the water resources of Yangtze River will be reduced by only 7% of its total amount.

Furthermore, in Northeastern part of China an inter-basin water transfer project from Songhuajiang River to Liaohe River will be implemented. The water amount transferred reaches to several billion m^3 and the diverting canal will be about 400 km long. It is called North-to-South Water Transfer Project in Northeastern China.

3.3 To Utilize All Kind of Water As Much As Possible

Owing to the relatively not high level of water amount per capita and per unit area of cultivated land to utilize all kind of water as much as possible is one of the effective ways for increasing available water. Here are included mainly saline water, nitric groundwater, water of hyper concentration of sediment, sewage water and so on.

Irrigation with saline water, including drainage water and return flow water in some cases, has been carried out with success in some provinces in North China and Northwestern China, such as Ninshia, Gansu, Inner Mongolia, Shanshi, Henan, Hebei etc. The salt concentration of water used reaches to more than 2 g/l, and the crop yield has been considerably increased due to increase of water supply for irrigation crops.

Irrigation with nitric groundwater is another way to increase water supply for irrigation. In these cases farmlands are irrigated by water with abundant nitric elements. It gives the irrigated land not only water but also fertilizer. Irrigation with nitric water has been successfully carried out in several northern provinces as well as in some regions in South China.

Irrigation with water of hyper concentration of sediment has been practised for a long time. Especially in Yellow River Basin irrigation with water of hyper concentration of sediment is being beneficial not only for increasing water supply for irrigation but also for reclamation of low yield farmland and flood protection. Recently the highest value of concentration of sediment in water diverted for irrigation farmlands amounts to 957 kg/m^3. In the case of using water with hyper concentration of sediment the key problem to which more attention must be paid is to keep the canal system in good operation conditions. For this purpose a wide variety of engineering measures have been adopted successfully. The history of utilizing water with hyper concentration of sediment for irrigation can be traced back to three thousand years ago and in some regions it has a great vitality up to now.

Irrigation with sewage water is widely used in the suburbs of large and medium cities in China. It can increase water supply for irrigation considerably in these regions and then increase the yield of crops from the farmland irrigated, especially of vegetables.

3.4 To Keep the Environmentally Sound Water Resources Utilization

Due to national economy development there has been an increasing important problem about environmentally sound utilization of water resources. Waste water from industry, agriculture, as well as domestic uses increases rapidly day by day. According to incomplete statistics the total discharge of sewage water of the whole country amounts to 36.8 billion tons every day in 1988, among which 26.8 billion tons are from industry. Most part of this large quantity of sewage water is retarded directly into various water receptacles without any treatment. 432 rivers were polluted in different degree based on environment monitoring of 532 rivers. It leads to degradation of quality of available water and shortage of fresh water for use, especially for irrigation of farmland and drinking of human being and livestock. At the same time pollution of groundwater in urban regions has been developed in a wide spread suburb regions of the large cities.

In order to utilize the water resources as full as possible, it is necessary to investigate the environmental impacts of the water resources engineering structures, especially the large reservoirs, inter-basin water transfer projects and large irrigation and drainage systems, and protect the water resources out of pollution.

4. SUMMARY AND CONCLUSIONS

Water resources have been distributed inequitably in the different regions of the world. More and more countries will be facing the water crisis in the next century, especially developing countries, such as the countries in Africa, Near East, Asia and North America etc.

China is one of the developing countries in the world. There is considerable amount of natural resources, which support the life of a huge population and the development of the society. The annual mean amount of water resources in China is about 2700 billion m^3, which takes the sixth place in the world in descending order, but water resources in China can not be considered as abundant, because the per capita amount of water resources is only about 2600 m^3, which is approximately 1/4 of the world mean, and the available water amount for unit area of cultivated land is about 80% of the world mean.

Due to development of national economy and improvement of human life the annual water demand in China during the second half of this century will increase by about 7 times in comparison with that at the middle of this century.

Consequently, there will be serious contradiction between the water demand and the available water. The possible ways for meeting the water demand in future could be grouped into two basic categories: to save water demand as much as possible and to utilize the water resources as fully as possible.

The strategic measures of the first group could be pointed out water saving measures for irrigation, which is the largest water user in the economy development, such as canal lining, application of low pressure pipeline network, application of water saving irrigation techniques, improvement of irrigation system operation and so forth.

There are also great potentials for water saving in industry and for domestic purpose as well.

The strategic measures of the second group could be pointed out as the following aspects.

1. To construct water storage facilities in the appropriate locations for regulating the uneven distribution of water resources in time in order to utilize them as fully as possible.

2. To develop inter-basin water transfer in various size in different regions in order to regulate the unequitable distribution of water resources in space for full use of them.

3. To utilize all kinds of available water including the reuse of water such as drainage water and sewage water as much as possible.

4. To maintain the use of water resources in environmentally sound conditions in order to avoid degradation of water quality.

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WATER DEMAND AND MANAGEMENT IN WATER DEFICIT AREA - A CASE STUDY

R.K. Sivanappan[1]

ABSTRACT

Tamil Nadu state is one of the water deficit states in India. The demand of water for agriculture, industries and municipal needs is increasing day by day due to increased population, but the entire available surface water and more than 60% of the ground water have already been harnessed and used.

This paper details/assesses the potential of the available surface and ground water in the state based on the studies made by many authors and agencies the demand of water for agriculture, industry and municipal needs for the year 2001/2010.

The gap between the available and demand of water for various purpose is about 1.75 million hectare meter (MHM). Which is 23% of the available potential. How to manage the gap? Though India is blessed with substainal water potential, but it is difficult to divert the excess water from one state to the other states.

This paper details the short and long term measures to be followed, policy options to be adopted by the government, farmers and others including research and development activities. Establishing a technology mission for water is suggested for the state. By implementing the various measures, it will be possible to overcome the problem of water scarcity/demand in the state.

1. INTRODUCTION

In nature, the traditional source of water supply namely surface and ground water source are inequitably distributed among the human population and locations resulting in social and economic imbalances. The per capita availability of water for the world is about 7000-8000 m^3, whereas, it is only about 1500 m^3 for India. With the rapid growth of population, even this amount will dwindle further. Tamil Nadu is one of the water deficit states in India. The average rainfall in Tamil Nadu is only 948 mm compared to India's average of 1250 mm. The population of this state is about 7%; the land area is about 4% and the water availability is only 3% compared to India's population, land area and water resources. Further the density of population is 430 compared to India's 250. Even during normal years, Tamil Nadu is facing water shortage/deficit.

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The population of the state is 55.64 million and the total geogrophical area of the state is 13 MHa of which 6.5 MHa is cultivated. The gross irrigated area may vary from 2.60 to 3.60 MHa depending upon the rainfall and the amount of water received from the neighbouring states and inter basin transfers. Though the annual rainfall is 948 mm, which is erratic and not uniformly distributed and in pockets in some years the amount of rainfall is reduced drastically. The state has harnesed more than 95% of the surface water resources and about 50-60% of the ground water resources through 1.6 million wells in the state. The details of the area irrigated and sources are given in table 1.

Table 1. Area irrigated - sources wise in Tamil Nadu in 1000 ha

Source	84-85	88-89
I Net area irrigated by		
Government canal	896	810
Tanks	715	479
Wells including tubewell	1007	1071
Other sources	21	14
II Area irrigated more than once in the same year	866	498
III Gross area irrigated	3506	2873
IV Irrigation intensity (%)	132.8	121
V Gross area irrigated to gross area sown	49.5	44.5

Source - An economic appraisal (1991), Government of Tamil Nadu.

The demand of water is increasing day by day not only for agriculture, but also for industries and drinking and other municipal needs for the growing population.

It is estimated that the water used for agriculture is more than 90% and this figure will come down to 75 to 80% in 2000/2010. The drinking and municipal use will increase since the government is providing water supply facilities to all the villages and towns. The quantity prescribed is about 80 LPd per capita where the population is less than 1 lakh (0.1m) and 100 LPd per capita where the population is more than 1 lakh. Many industries are established in the recent past and many more will be provided in the next 10-20 years especially after the liberalisation of industrial policy in the country.

2. WATER RESOURCES POTENTIAL

2.1 Surface water

The water is available from i) Rivers/streams as surface water and ii) Wells (open and tube wells) as ground water. The source for the water is only rainfall. The total rainfall over Tamil Nadu is about 12 MHM per year. The surface water availability is calculated by various authors/agencies. The utilisable surfaces flow according to Kumaraswamy (1974) is about 2.486 MHM. According to Irrigation Commission (1972), the annual surface flow in Rivers of Tamil Nadu is estimated at 3.18 MHM. The National Commission on Agriculture (1976) has estimated it as 2.55 MHM. The surface water availability is taken as 2.5 MHM.

2.2 Ground water

The Ground Water Potential of Tamil Nadu is estimated by various scientists and agencies in the last 30 years. The details are given in Table 2.

Table 2. Ground water potential assessment in MHM

Author/ Agency	Ground water potential	Utilised	Available
1. Sakthivadivel (1974)	1.78	0.90	0.89
2. Irrigation Commission (1972)	1.42	--	--
3. Task force on Ground water (1972)	1.40	--	--
4. Work group on the estimation of ground water (1992)	2.23	1.36	0.87

Though the figures are varying from 1.40 to 2.23 MHM, the potential can be taken as 2.00 MHM taking the average estimates of Dr. Sakthivadivel (1974) and the work group on the estimate of ground water (1992).

2.3 Water from other states and inter basin transfers

Besides the above two sources, there is also to some extent, the import of surface water including inter basin transfer from neighbouring states of Andhra Pradesh, Karnataka and

Kerala. It was roughly assessed as 1.50 MHM of water, but this quantity has been reduced since there are many problems in sharing waters with the respective state governments. The imported/transported water may be taken as 1.0 MHM. Therefore the total water potential for Tamil Nadu would be about 2.50 + 2.00 + 1.00 = 5.50 MHM. Perhaps this estimate also requires further debate and refinement.

3. DEMAND FOR WATER

As more and more water is needed for agriculture, industry and public for high standard of living what was once enough is not so any more with consequent increasing demand. Water is no more a free good, rather it is an economic good. Estimating water demand, particularly projections into the future poses serious problems. Future projection are uncertain as they involve estimates of population growth and economic activity levels in various sectors. However, taking into account the basic need regarding the future agricultural growth; with respect to areas under irrigated crops, population growth, livestock, industrial development, recreation, power generation, etc., an estimate can be made.

3.1 Demand for non agricultural sector

The domestic and industrial uses of water and other non agricultural sector at present claim a share of about 10% and this is likely to be increased to 25 to 30% with increased industrialisation, urbanisation and growing needs of rural areas for improved facilites for drinking water as per the study of Swant (1980). The National Commission on agriculture has estimated that by 2000 AD, the requirement of fresh water for non agricultural purposes may be about 27% of the available fresh water. A growth rate of eight percent for industrial production was recommended at the (1979) futurology workshop on water needs and management for working out the industrial sector needs. For municipal needs the required quantity of water is 100 LPd per person if the town/city is having population more than 100000 and 80 LPd for population less than 100000. In respect of livestock population and consumption of water it is pointed out that though the area intensity of cultivation & irrigated area are likely to increase, it is assumed that the livestock population which is already more may have to remain at the present level only.

The total demand for non agricultural purpose is estimated as follows according to Sivanappan & Palaniswamy (1982):

i) Industry (25% of present supply)	1.25 MHM
ii) Population needs house hold/schools	0.30 MHM
iii) Live stock	0.10 MHM
iv) Other needs	0.10 MHM
Total	1.75 MHM

3.2 Demand for agriculture

Agricultural sector is consuming about 90% of the available water resources at present. Much attention should be given on the agricultural water needs, its present and future utilisation pattern etc. As reliable information regarding actual water use is not available, the probable utilisation and demand pattern is derived based on i) growth rate of irrigated crops and ii) irrigation requirements as adopted by National Commission on Agriculture/Irrigation Commission.

3.2.1 Growth rate of irrigated crops

The compounded growth rates of area under irrigated crops were worked and projected for 2000/2010 AD. The projected area and water requirements are given in Table 3. It is seen from the table that the projected area to be irrigated is about 5.27 MHa and the corresponding water requirement is about 5.00 MHM which is higher than the supply available for agricultural purposes i.e., 3.75 MHM (5.50 - 1.75 = 3.75 MHM). The demand for and supply of water in 2000/2010 is given in table 4. The suggested water requirements by National Commission/ Irrigation Commission may be alright for the country as a whole, but it should be higher for this state since 70% of the present irrigated area and about 50% of the projected irrigated area require more than 1500 mm of water. Hence the growth rate figure of 4.88 MHM or 5 MHM is taken as the demand for water for agriculture purposes.

4. SUPPLY AND DEMAND GAP FOR WATER

The increasing demand for water over supply of water is presented as the supply demand gap for water. The supply demand gap is about 1.25 MHM (23%). It is important to bridge the supply demand gap either by reducing the demand or by increasing the supply level to match the growing demand in future. Though excess water is available in other states, it will be difficult to divert in the immediate future and hence management of the available water is crucial to further the economic growth and development for the water deficit areas like Tamil Nadu. Since the gap between demand and available water is substantial, planned long term and short term measures, research and development activities and implementing policy options for the government and farmers are very essential to overcome the problems.

Table 3. Growth in irrigated area and water requirements in 2000 AD in Tamil Nadu.

S.No.	Crop	Projected area in MHa	Water requirement in mm.	Projected water requirement in MHM
1.	Paddy	1.70*	1500	2.55
2.	Sugarcane	0.31	1750	0.54
3.	Banana	0.15	1500	0.23
4.	Cotton	0.50	600	0.30
5.	Groundnut	0.35	600	0.21
6.	Ragi	0.22	400	0.08
7.	Sorghum	0.25	400	0.10
8.	Bajra	0.14	400	0.04
9.	Pulses	0.27	250	0.07
10.	Maize	0.29	400	0.12
11.	Chillies	0.09	700	0.06
12.	Vegetables and Fruits	0.16	750	0.12
13.	Fodder crops	0.44	500	0.22
14.	Other crops	0.40	600	0.24
		5.27		4.88 or 5.00

(This is the area projected by National Commission on Agriculture - 1976)

* In the case of paddy crop area, it is assumed that by improved technology and water management practices, it would be possible to reduce the area under paddy; but at the sametime increasing the yield by two fold thus maintaining the overall production level.

5. WATER DEMAND MANAGEMENT

Since more than 90% of the available water is used for irrigation, high priority should be given for better water managements and increasing the water use efficiency including adopting advanced irrigation methods and change of crop and cropping pattern. The following are some of the methods which can be introduced to overcome the increasing demand for water.

5.1 General

There are numerous ways and means by which the demand can be met with. The measures are grouped into four categories.

Table 4. Demand for and supply of water in 2000 AD in Tamil Nadu.

S.No.	Particulars	Quantity in MHM
1.	Total water supply	5.50
2.	Demand for non-agricultural purposes	1.75
3.	Balance supply available for agricultural purposes	3.75
4.	Demand for water agricultural purposes based on:	
	i. Growth rate of crops	5.00
	ii. National Commission on Agriculture estimate[1]	4.22
	iii. National Commission on Agriculture revised estimate[2]	4.48

[1] Based on water requirement of 0.80 hectare metre per cropped hectare

[2] Based on water requirement of 0.85 hectare metre per cropped hectare

5.1.1 Short term measures

i) Better water management practices in canal, tank, well irrigated areas and in dry zones

ii) Introducing canal modernisation and on farm development warkes in canal and tank command areas to increase the water use efficiency.

iii) Introducing advanced methods of irrigation like sprinkler and drip to save water and increase the area

iv) Conveyance of water through pipes to avoid seepage loss

v) Crop diversication to get more profit from unit water

vi) Water conservation and harvesting techniques

vii) Water quality management and reuse of waste water

viii) Rehabilitation of irrigation tanks

ix) Salt water utilisation for agriculture

x) Training extension staff and farmers on water management

xi) Drainage improvement

xii) Farmers participation and management

5.1.2 Long term measures

i) Diversion of west flowing rivers to east (Kerala to Tamil Nadu)

ii) Linking northern rivers to south to start with all the peninsular rivers - Mahanadhi, Godavari & Krishna to Cauvery and Vaigai

iii) Collection and storage of rain and flood water

iv) Providing link canal from north to south parallel to the east coast to about 50-80 KM from the sea to feed the tanks

v) Cloud seeding in areas where it is successfull

vi) Converssion of sea water near the coast for drinking and industrial purposes.

5.1.3 Research and development

i) Establishment of Command Area Development Agency (CADA) for all projects

ii) Establishment of tank management agency/authority

iii) Formation of water conservation, ground water recharge and water harvesting organisation at Block, District and State level

iv) Technology mission on irrigation water management

v) Accurate assessment of water resources both surface and ground water

vi) Research programmes in water use and management

5.1.4 Policy options

i) Seperate cadre for irrigation/water management activities - i.e., seperate department

ii) Water management extension officer at block level

iii) Linkages between land and water to use effectively and efficiently

iv) Water should be considered as a economic good and not free

v) Water as a national property and plan to use it optimally

5.2 Thrust areas

Though there are many methods under various categories that have been identified, the thrust areas which can be immediately introduced/implemented are detailed below (Sivanappan 1994):

i) Crop diversification

ii) Introducing advanced method of irrigation like drip and sprinkler

iii) Involving farmers/farmers participation in water management

iv) Research technology transfer and coordination

5.2.1 Crop diversification

Paddy is the main crop which consumes more water. It is estimated that more than 80% of the water diverted for agriculture is used for growing paddy. The average yield of paddy is only about 4-5T/Ha whereas its potential yield is more than 10 T/Ha and some farmers have taken a maximum yield of 15-18 T/Ha. Therefore it is time to change the crop/cropping pattern not only to save water but also cultivate less water using crops like fruits, vegetables, flowers and other commercial crops which can be exported. It is time now to take up this challenge and plan for diversifying cropping pattern especially compatable in canal and tank command areas.

5.2.2 Introducing advanced method of irrigation like drip and sprinklers

In the tank and canal systems, the water is conveyed through unlined earthern channel and there is no effective control of irrigation water. It is suggested that sprinkler can be used for all closely spaced crops except rice. About 30 to 40% of water can be saved by this.

About 40% of the irrigated area is under well irrigation, micro irrigation (drip and mini sprinkler/cane wall/bi wall) which is suitable for all row crops, especially for wide spaced and high value crops can be introduced in these areas. Studies have indicated that about 50 to 70% of water could be saved and the yield of the crop is invariably more upto 100% (Table 5). By introducing this method, it is possible to maintain the ground water table at a reasonable level and at the same time farmer can get good income from the scarce water.

5.2.3 Involving farmers/farmers participation in water management

The technical ills of irrigation management have been widely discussed in recent years. They range from inadequate design and operation of main system to poor irrigation layouts and practices at the farm level. The adverse effects are well known to all. There is a rapidly growing awareness that the farming community must be involved in all phases of planning, design, implementation, operation and maintenance is good standards of water

Table 5. Water used and yield for various crops in drip and conventional mehtods.

S.No.	Crops	Yield (Qt/Ha)			Water Supplied (CM)		
		Conventional	Drip	Increase in yield (%)	Conventional	Drip	Water Saving (%)
1.	Banana	575.00	875.00	52	176.00	97.00	45
2.	Grapes	264.00	325.00	23	53.20	27.80	48
3.	Mosambi ('000 No)	100.00	150.00	50	166.00	64.00	61
4.	Pomegranate (000 No)	55.00	109.00	98	144.00	78.50	45
5.	Sugarcane	1280.00	1700.00	33	215.00	94.00	56
6.	Tomato	320.00	480.00	50	30.00	18.40	39
7.	Watermelon	240.00	450.00	88	33.00	21.00	36
8.	Cotton	23.30	29.50	27	89.53	42.00	53
9.	Ladies Finger	152.61	177.24	16	53.68	32.44	40
10.	Brinjal	280.00	320.00	14	90.00	42.00	53
11.	Bitter Gourd	154.34	214.71	39	24.50	11.55	53
12.	Ridge Gourd	171.30	200.60	17	42.00	17.20	59
13.	Cabbage	195.80	200.00	2	66.00	26.67	60
14.	Papaya	134.00	234.80	75	228.00	73.30	68
15.	Raddish	70.45	71.86	2	46.41	10.81	77
16.	Beetroot	45.71	48.87	7	88.71	17.73	79
17.	Chillies	42.33	60.88	44	109.71	41.77	62
18.	Sweet Potato	42.44	58.88	39	63.14	25.20	60

Source : National Committee on the use of plastic in agriculture (NCPA) 1990.

management are to be achieved. The involve farmers, there is a need to create water users organisation as sluice level, canal level and as the projict level in tank and canal irrigation projects. In the Lower Bhavani Irrigation Project, Tamil Nadu, farmers association at micro (sluice) and macro level (project) are being organised to manage the water efficiently by themselves.

5.2.4 Research transfer of technology and coordination

To cope up with the demand/to bridge the gap, the following suggestions are made:

i) Production increases are to be achieved through optimal utilisation by taking up studies on land water - fertiliser production functions.

ii) New technologies have to be developed for maximising water use in different situations

iii) Planning and technology transfer for increase land and water use strategies in the regions

iv) Human resource development by way of training and visit

Further, there is no effective co-ordination between scientists and extension workers and also among extension workers of different departments/disciplines. Further it is suggested to create a technology mission on irrigation water management for the state to pay integrated attention for using the available water in the most beneficial manner to increase the productivity for the state and profitability to the farmers.

6. SUMMARY AND CONCLUSIONS

Tamil Nadu is a water deficit state in India. The demand for water is increasing day by day. Since the water potential of Tamil Nadu is limited and almost fully utilised at present, the increased demand for the coming years is to be met only by introducing advanced techniques in water conservation and management. The water potential including import of water from neighbouring state is about 5.50 MHM. The total demand of water in 2000/2010 AD is estimated about 5.00 + 1.75 = 6.75 MHM. The demand for agriculture is based on the actual requirements of water for various crops taking into account the water management research findings. The demand for non agricultural use i.e., for industries and municipal needs are based on the growth needs of industries and population.

The gap between the demand and available water resources is 6.75 - 5.50 = 1.25 MHM, is substantial (about 23%). The gap can be met with by taking many measures like short term, long term, research and development and by adopting some policy options. The very important measures are to follow water management practices, introducing advanced method of irrigation like drip and sprinkler, changing crop and cropping pattern, creating agencies to take up conservation, storage, distribution and management of water resources. It includes forming technology mission for water, taking up research on water and following water policies in the state. It is possible to meet the challenges if adequate action is initiated immediately. In the long run, water can be diverted from surplus basins of north and west flowing rivers to this state. By implementing the above measures/suggestions, it will be possible to overcome the gap between the demand and available water resources of the state in the coming years. The same strategy can be adopted in may other water deficit states in India and other developing countries to solve the water demand problems in the future.

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ISSUES TO ADDRESS FOR RELIEVING PROBLEMS OF WATER SCARCITY IN DROUGHT CONDITIONS

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ABSTRACT

Water scarcity in arid regions with long drought periods brings suffering to increasing number of people. The development of integrated approach and methods of non-polluting and multiple water reuse, becomes of paramount importance in such regions. Dry sanitation is a viable and economic option possible to be fully implemented in any country, domestic "grey water" may be used several times after biological treatment in very efficient root-zone constructed facilities and sand filters. Progress in biotechnology makes it possible to design biological self-sustainable systems able to purify any water by removing toxic or unwanted substances from any effluent. Harvesting and safe storage of rainwater, dew and fog are the other important research topics that should find attention of funding organizations. Problems of competing water needs of urban and rural areas may be relieved by changing the basic approach to sanitation and drainage issues in urban areas. Wastewater and stormwater from urban areas may be treated to higher standards, its resource value (carbon, nitrogen, phosphorous) compensate costs of higher treatment. Urban constructed wetlands, storages, treatment trenches installed locally provide water supply, bring flood mitigation, pollution control, simultaneously providing high revenues in relation to capital costs. With respect to arid and semi-arid regions, the ability to look at the problems in a different perspective may be very valuable.

INTRODUCTION

Water scarcity in arid regions with long drought periods brings suffering to increasing number of people. It can be argued that water quality issues have taken over too large a share of attention of funding organizations and research potential. This opinion is justified only when issues of sanitation and drainage are considered separately from water supply issues. However, the less water available, and the longer dry periods, there is the more need for an integrated approach. The development of methods of non-polluting water use in any human activity and multiple water reuse, becomes of paramount importance in such regions.

The challenge for the next decades will be to introduce innovative water technologies, management systems and institutional arrangements which are able to meet the multiple objectives of satisfying water demands, economic efficiency, equity, environmental integrity, simultaneously maintaining or/and providing high level of water services for the residents of rural and urban areas.

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DRY SANITATION

Dry sanitation is a viable and economic option possible to be fully implemented in any country, not only releasing significant amounts of water but also bringing possibility of recycling carbon and nutrients for bio-mass production. Domestic water ("grey water") may be used several times after biological treatment in simple, and in a warm climate very efficient root-zone constructed facilities and sand filters (EPA 1998, Mann 1990).

Another option available in future scenarios is the most appealing one: no wastewater as such is produced at all in urban areas: dry composting toilets are installed in each house, compost used in agriculture, drinking water is not used in toilets (really it is a nonsense to do it). Besides clear environmental benefits, there are economical benefits to gain from such a solution. Following example may illustrate such statement: Cost of treatment in traditional treatment plant is: ca 90 SEK for removal of 1kg Phosphorous and ca 300 SEK for removal of 1kg of Nitrogen. Counting with reduced treatment costs plus value of composted excrements as fertilizer benefit is 465 SEK/person and year i.e. 3,95 billions/year in Sweden (Gunther 1992).

Calculation for one existing village, 100 families, 250 persons: Total reconstruction cost to separation toilets (urine separate - used directly as fertilizer, excrements composted and used in agriculture) is 1,58 ml SEK, "grey water" (all water without toilets) is treated in small root-zone facilities with a surface $17m^2$ / person - construction cost 0,5 ml SEK - total cost of change from traditional to novel: 2.1 ml SEK minus value of fertilizer (urine and faeces) to compare with 5 ml SEK for pipes to nearest treatment plant plus treatment costs ca 100.000 SEK per year. On top of such calculation comes possibility of reuse of grey water and reduction of water consumption. Intangible environmental benefits not counted. Now, people will say that water closet is a top of luxury. This firm belief may be seen as an obstacle in progress. However, in Sweden there are several "old" eco-villages with dry toilets, on hundreds of islands around Stockholm dry toilets are used for decades. Recent inquiry among children born and raised in dry-toilet-houses shown that they clearly dislike water toilets for three reasons: the smell, the splash and the look of "it"; these children are disgusted when using water toilets, contrary to the dry composting toilets, does not produce these three unwanted effects! It is a very nice prove that our sense of luxury and habits can change during only one generation.

Though there is a lot of practical experience about the construction, economy, operation and maintenance of such facilities in temperate climate further research and demonstration projects in arid countries should be intensified. There is a number of successful application examples in so called "ecological villages" where high degree of recycling of water is achieved. Below follows description of several successfully operating ecological villages in Sweden.

"Ecological villages" in Sweden, examples:

Toarp, Malmö Commune:

Construction year: 1990
Number of houses: 28 + common nursery (single and two-family houses, ca 100 persons totally).
Water: All armature is water saving type. Water use 70 l/person day on the average. (Typical use in Sweden 140-200 l/person day). Drinking water from own deep wall. "Grey water" (kitchen, bathroom, laundry) is mixed with stormwater, collected in central tank, pumped up and treated in three steps: root-zone, sand filter, pond. Released to the river.
Toilets: Composting (Snurredass) emptying twice a year. Compost is used small agriculture. All residents own a peace of land. Underground cellar is available for all households.
Energy: Double insulation of houses. Heating using sun-panels on roofs, additional wood stoves. Recirculation of heat, heat exchanger. Central vacuum cleaner in each house.
Water and sewage charges: reduced.
Comments: No problems, good standard of living, no fluid pollution out. Solid wastes separated at each house.

Tugelite, Karlstad Commune:

Construction year: 1983
Number of houses: 16 + common nursery, washing center
Water: All armature is water saving type. Water use 70 l/person day on the average. (Typical use in Sweden 140-200 l/person day). "Grey water" (kitchen, bathroom, laundry) is collected in each house, after primary sedimentation is used for irrigation of small agriculture. Surplus is released to communal sewage network.
Toilets: composting (Snurredass). Compost is used in agriculture.
Energy: Double insulation of houses. Heating using sun-panels on roofs, additional wood stoves.
Water and sewage charges: reduced.
Comments: no problems, life-style of importance for success.

Åkesta, Västerås Commune:

Construction year: 1990
Number of houses: 28
Drinking water: from own deep-wall. Necessary to de-ironize.
Water: reduced use. "Grey water" (kitchen, bathroom, laundry) is collected in each house, after primary sedimentation treated in a sand filter and used for irrigation.
Sedimentation tank 25 m^3, Pumping well 2.5 m^3, Sand filter capacity 18 m^3/day, sand filter deep 1m, surface 500 m^2.
Toilets: Wolgast type: separating urine from solids. Urine is collected in each house and lead to a central tank. Urine is given free to the farmer. Compost from 5 composted solids is used in agriculture.
Energy: traditional.
Water and sewage charges: Reduced. Emptying of tanks 200 Skr/house year.

Comments: no problems, it is nice to have toilets without smell. Odor problem from sand filter, resolved after reconstruction.

Dalby "Solby", Lund Commune

Construction year: 1989
Number of houses: ca 30
Drinking water: from communal network.
Water: reduced use. "Grey water" (kitchen, bathroom, laundry) connected to communal sewage network.
Toilets: Composting, type Snuredass and Multrum. Additionally water toilets in large apartments. Compost is used in agriculture.
Energy: traditional.
Water and sewage charges: Reduced.
Comments: Containers for Multrum toilets designed too small, function better after introduction of worms. Size of chamber with containers should be bigger to make emptying easier. 40% of residents use water closets only.

STORM- AND WASTEWATER TREATMENT BY SOIL-AQUIFER INFILTRATION SYSTEMS

Wastewater and stormwater from urban areas may be reused also indirectly after treatment by soil infiltration or/and percolation to aquifers (Soil-Aquifer Treatment -SAT, Bouevr 1991). This technique brings several benefits compared with traditional treatment: 1) wastewater is treated to the high standards, 2) groundwater depletion is avoided, 3) surplus of water delivered to the aquifer may be used for example for irrigation, 4) saltwater intrusion in coastal areas may be avoided. Cost of soil infiltration treatment are reduced by a factor of 2 to 10 compared with the costs of traditional treatment. Below follows description of some Swedish experiences with soil treatment of storm- and wastewater:

Wastewater treatment by soil/sand filters:

According to Swedish Environmental Protection Board (SNV 1985), between 1981 and 1983, more than 14000 waste-water infiltration facilities have been constructed for single family houses, for settlements with more than 25 persons, have totally 706 larger infiltration facilities been constructed until 1983. It is stated that infiltration to the ground gives at least 50 per cent phosphorous reduction. Reduction of coli- forms and pathogens in an infiltration bed of 50-80 cm thick is of the same order of magnitude as in active sludge treatment, i.e. ca 99.9 %. Construction and operation of facilities is technically simple but require extensive experience and knowledge in order to ensure optimal operation.

1993 there is about 1000 soil/sand-filter facilities for wastewater treatment in Sweden. Typical size about 1000-4000 persons. Largest facility for ca 5000 persons. Problems only in first 100 constructions. In newer constructions no major problems observed. Good treatment effect, according or above regulations. Costs between 1/2 and 1/10 of traditional treatment.

Stormwater infiltration:

Inventory of stormwater infiltration facilities in nine communities in Sweden and studies of long-term function and maintenance problems in 11 oldest facilities in Nordic countries were performed in 1981. Three main types of stormwater infiltration facilities are in use: surface infiltration, ditch infiltration and percolation basins. The oldest facility in Sweden has been in operation for 22 years, however most of them is not older than 15 years. The areas occupied by infiltration surfaces is varying between 0.5 and 100 ha with an average of about 20 ha.

Stormwater treatment in constructed wetlands:

Toftanäs, Malmö Commune:
Construction year: 1989
Impermeable area connected: 190 ha (industrial and residential area).
Wetland area: 2 ha
Construction: Periodically flooded pond with several levels. Meandering course of flow. Ca 300 plant species used in treatment.
Function: Energy dissipation, retention, sedimentation, treatment in rootzone on differentiated depth. Reduction of N ca 50 %.
Costs: ca 4 ml Skr less than traditional system.
Comments: Good treatment effect, increased with age of the pond. Recreational value, preservation of natural flora, shelter for many birds. Good effects can be achieved by combining ponds, ditches, root zones and wetlands in one system.

VALIDITY FOR OTHER CLIMATIC REGIONS ?

One can argue that given examples of treatment an re-use of wastewater from Sweden are not relevant for other climatic regions. However, though these experiences are, of course, not directly valid, it is worth to notice that the treatment processes in all such facilities are essentially the same as in any traditional treatment plant, i.e. solubilization of solids, aerobic and anaerobic bacterial decomposition processes. Since biological activity is only more efficient in a warm climate, it should be possible to translate the results into terms of applicability in arid climate. This requires lunching of research and demonstration projects.

INTEGRATING FARMING SYSTEMS

Progress in biotechnology makes it possible to design biological "starters" initiating self-sustainable bio-systems able to purify any water by removing toxic or unwanted substances from any effluent. The concept of Integrated Farming System (IFS) is a modern modification of ancient technology used for thousands of years in many Asian countries. Such systems were, regretfully, in many places abandoned to give place industrialization in the name of development (Zhang and Qian 1991, Jisong 1991). IFS combines agriculture and its wastes with aquaculture and agro-industry. The target is to utilize all nutrients, other organic material and water by reinforcement of microbial symbiotic interaction. All residuals from one process are imputed to the other one. IFS contains four stages of treatment-production: 1) Simple digesters: biogas production and reduction of BOD by ca 60 % in simple digesters. 2) Shallow algae basins: further reduction of BOD by ca 30 %, fixation of CO_2, production of oxygen using algae as high-protein feed. Algae grows ca 20g dry matter/m^2 day. 3) Deep fish ponds: intensive growth of plankton and autorotrophics eaten by fish. Fish yield is highest in ponds 2.5 - 3 m deep. Heterotrophics present in lower part of the pond produce high-protein feed for higher fishes. Culture for aquatic food plants. Source of irrigation water. 4) Various planting media: multi-cropping, rotation, harvesting. Species used: silkleaves, fruits, mushrooms etc (Chan 1993).

One can ask the question why such a systems are not used in a larger extent in humid tropical countries. The reason can probably be found in a fact that development process in these countries proceeds according lines of development in high industrialized countries, no matter differences in climate, social structure, traditions etc. At present, several traditional water-related technologies used in industrialized countries has shown to be not very wise. More and more voices advocate a need of changing these technologies and general approach. There is no doubt that solution of several present problems, will be found by application of technologies that are not high-tech in a sense of built-in resources and machinery but which contain very high degree of know-how and bio-technology related knowledge combined with basic principles of preventive approach.

Application of IFS in semi-arid regions is investigated and possibly will be used in conjunction with see-water irrigation or/and see-water farming. One conclusion may be formulated however: there is an obvious connection between our approach to wastewater and the wellbeing of the population. Resources present in wastewater may either be wasted bringing environmental degradation or used as a valuable source of nourishment for the growing population.

These developments should be, again, translated into viable options matching the needs of regions experiencing drought conditions.

STORMWATER

Consider volume of water that is delivered to the urban areas by the nature in a form of rainfall: 100 mm rain on 1 km^2 impermeable area gives 100 000 m^3 water i.e. enough to 1830 people during one year counting with water use 150 l/day. If dry toilets are used, water consumption of households may be reduced by 70 %, that is 5500 people may gain all necessary water from 1 km^2 and rainfall 100 mm/year. With other words, theoretically, 182 m^2 impermeable area can deliver water that 1 person needs. This amount of water is considerable, and though in practice it cannot be easily utilized, it should be considered as an important resource. Utilization of this water require basic change in applied technology of stormwater management. Western traditional technology, i.e. pipes for fast removal of stormwater from urban areas is developed for wet climate countries. Semi-arid and arid countries make a mistake copying this technology.

RAINWATER HARVESTING

Harvesting and safe storage of rainwater, dew and fog are the other important research topics that should find attention of funding organizations. Periodic shortage of drinking water can be, amazingly, observed not only in arid regions but also in several parts od the Swedish archipelago. In order to ensure a constant supply of fresh water there is a need to store water on seasonal as well as yearly basis.

The method of collection, storage and conservation of rainwater was developed by the Swedish inventor Karl Dunkers. The method eliminates some of the shortcomings associated with the traditional way of storing water, thus promising a solution of acute water shortage problems in drought regions. The reservoir can be situated on land or at sea. Storage on land is traditional and contains tanks and treatment units. Storage at sea consists of floating tanks with flexible sides. Due to the difference in density between fresh and salt water, the tank has no bottom, fresh water is floating on top of salt water. This idea may have a development potential in coastal zones of arid countries (Hogland, Niemczynowicz and Widarson 1986).

INSTITUTIONAL CONSIDERATIONS

Present sectorized institutional and organizational structures are often unable to carry on actions integrating urban sector dealing with water supply, sanitation and drainage, and rural sector dealing with development of water resources for irrigation and rural use. Problems of competing water needs of urban and rural areas may be relieved by changing the basic approach to sanitation and drainage issues in urban areas. There are new ways of thinking leading us to the conclusion that it may be possible to treat wastewater and stormwater from urban areas to higher standards, simultaneously achieving economical benefits compared with traditional solutions, because the resource value of wastewater (carbon, nitrogen, phosphorous) compensate costs of higher treatment. In some areas (example Perth, Australia), sufficient water volumes can be generated from wastewater and stormwater within the urban area; it will be no need to import any water to urban areas. Urban constructed wetlands, storages, treatment trenches installed locally provide water supply, bring flood mitigation, pollution control, simultaneously providing high revenues in relation to capital costs (Boisen 1991, Cooper and Findleter 1990). Removal of uniform pricing and standardization is seen as a necessary condition to introduce incentives, efficiency improvements, and social equity considerations.

These ideas may seem a "science fiction" with respect to developing countries in arid and semi-arid regions, but their most essential element, i.e. ability to look at the problems in a different perspective may be very valuable in any climate conditions.

CONCLUSIONS - THE VISION

Taking the lesson from the nature we should act to increase self-organization ability of nature-human systems in order to create a viable structure capable of recycling resources. Water is the most precious resource of mankind. Recycling of water in dry lands is an only viable alternative for the future. This may be achieved by minimizing artificial (technical) manipulation and the use of energy. Simultaneously, social self-organization processes and the use of ecosystems as regulators should be maximized. Recent developments in eco-technology and bio-technology can provide the means that are necessary. There is no doubt that solution of several present problems, will be found by application of technologies that are not high-tech in a sense of built-in resources and machinery but which contain very high degree of know-how and bio-technology related knowledge combined with basic principles of preventive approach (Marti-Brown 1992, Niemczynowicz 1993).

The challenge for the next decades will be to introduce innovative water technologies, management systems and institutional arrangements which are able to meet the multiple objectives of equity, environmental integrity and economic efficiency, simultaneously maintaining or/and providing high level of water services for urban residents.

We may imagine future water-sensitive and ecologic cities where stormwater, after multiple reuse, flows on the surface in blue-green strokes of streams and lakes with high recreational value, and leaves the city still clean. The city is in perfect balance with the rural areas around. Wastewater treatment is performed in small highly specialized bio-systems where all solids are selectively taken out for reuse. Wastewater enters the system, clean water leaves the system, plus we have reusable resources such as wood (energy), paper, cattle food, chemicals, food, etc. All nutrients present in wastewater are not wasted but used in production of new biomass. Such systems have yet to be developed, but parts of them may be used to-day.

Basic changes are required not only in applied technologies but also in education systems, aid programs, social habits, policies, structure and management of the societies. To change all this is an immense task that will take decades. But formulation of the goals of this change, and a consensus about its spirit, are necessary in order to state the direction of future actions.

The basis for a new approach will need to include all of the following key elements:

* *Integrated system approach* with both structural and non-structural elements in contrast to narrow-minded technological approaches.
* *Cross-disciplinary cooperation* in solving complex problems.
* *Small scale* solutions in contrast to technological monumentalism.
* *Source control* instead of the "end of pipe" approach.
* *Local reuse and disposal* instead of exploitation and wastefulness.
* *Pollution prevention* instead of reacting to damage.
* *The use of bio-technology and ecological engineering* in wastewater and waste management (Niemczynowicz 1992, 1993).

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CONSTRUCTION MANAGEMENT : THE BOTTLE NECK FOR IRRIGATION IMPROVEMENT IN EGYPT

Abdel Fattah Metawie[1]

ABSTRACT

Egypt has launched a new Irrigation Improvement Project (IIP) in order to modernize its irrigation system.

The recommendation for improvements was based on a unique applied research project called Egypt Water Use and Management Project (EWUP). During the implementation stage of the IIP many lessons have been learned one of which is the need for construction management which is the bottle neck for water management program and which could accelerate the transfer of technology process both in the hardware and/or the software of the system.

The complexity of the irrigated agriculture system in Egypt necessitates training contractors who execute the projects.

Buying time during the implementation of the water resources projects is of vital importance. One way to look at it is through users' participation for high quality control. Results and experience in implementing land leveling program within the irrigated agriculture system in Egypt have shown many operational problems which could be overcome in implementing construction management program such as time constraints.

Users, operators, managers, planners and contractors are all parties which could gain both direct and indirect benefits through construction management; in this case, potential increase in crop production and the saved resources would increase capacities and abilities to cover the costs of the new system as well as drawing net benefits.

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1. INTRODUCTION

An intensive water management applied research program has been implemented in the Old Lands of the Valley and the Nile Delta of Egypt from the late seventies to the middle of the eighties.

The main objective of the project was to identify the problems as well as the constraints that hinder the effective use and the performance of the intensive irrigation and drainage network of canals and drains of the Nile system. Following the stage of problem identification and the search for solutions, an implementation program was performed in selected pilot areas to test solutions.

The recommendations of this unique and successful research project have been handed to the Ministry of Public Works and Water Resources (MPWWR) in Egypt, which has initiated the Irrigation Improvement Directorate in the middle of the eighties as an implementation agency of these recommendations and later to launch the Regional Irrigation Improvement Project (RIIP) in eight Governorate of Upper, Middle, and Lower Egypt.

Hopes were high in transforming the existing system to a modernized system. But change requires time and compatibility within the irrigation sector, as well as with other individuals and sectors involved either directly or indirectly in the irrigated agriculture system in Egypt.

The paper will discuss the construction management as one of many issues which constitutes a serious bottle neck for water management in order to accelerate the process of technology transfer of the new hardware and/or the software of the system. Background information about the development process of the irrigation improvements in Egypt as well as the transfer of technology problems will be discussed.

The need for construction management within the irrigated agriculture system necessitate users' participation in the process, contractors training, and the application of research results to the implemented projects which are important in order to maximize benefit during the implementation stage, obtain high quality control and increase the performance of both hardware and software of the new system.

2. STATEMENT OF THE PROBLEM AND THE POTENTIAL BENEFITS:

The Irrigation Improvement Project (IIP) has done an intensive monitoring and evaluation program in 1992-1993 whose findings made by the IIP Monitoring and Evaluation Unit and the Irrigation Advisory Service (IAS) Staff were published in early 1994.

The findings of the program have included benefits, constraints, and problems. For more progress in the period 1994 - 1995 the following is required.

1 - Legal base for WUAS and cost sharing with incentive policy.
2 - Construction management.
3 - Continuous flow policy.
4 - Finishing touches to mesqas.
5 - Implementing official mesqa turnovers from contractors to WUAS.
6 - Staffing turnovers in IAS.
7 - Training of staff and WUAS.
8 - On-farm water management.
9 - Documentation of IIP benefits.
10- Constant Public awareness and top-level commitment.

Since the paper is to discuss construction management, Figure (1) shows a list of contracts for 132 improved mesqas and the elapsed period for their completion against the contract time. It also shows the percentage of mesqas in each contract completed by February 15,1994. Construction of mesqas was new for most contractors in the beginning, but was soon acquired experience.

The IIP realized that the long delays in construction, the incompletion of the contracts and sometimes the poor quality of construction was most demoralizing to farmers.

When opposition to improvements emerged at an earlier stage the major cause was primarily the long delays in the completion of construction.

On the other hand, if one was to look at the measured and/or potential benefits of the IIP, the question which would be raised is why the project does not progress and resolve issues.

Table (1) summarizes some of the measured benefits of the project.

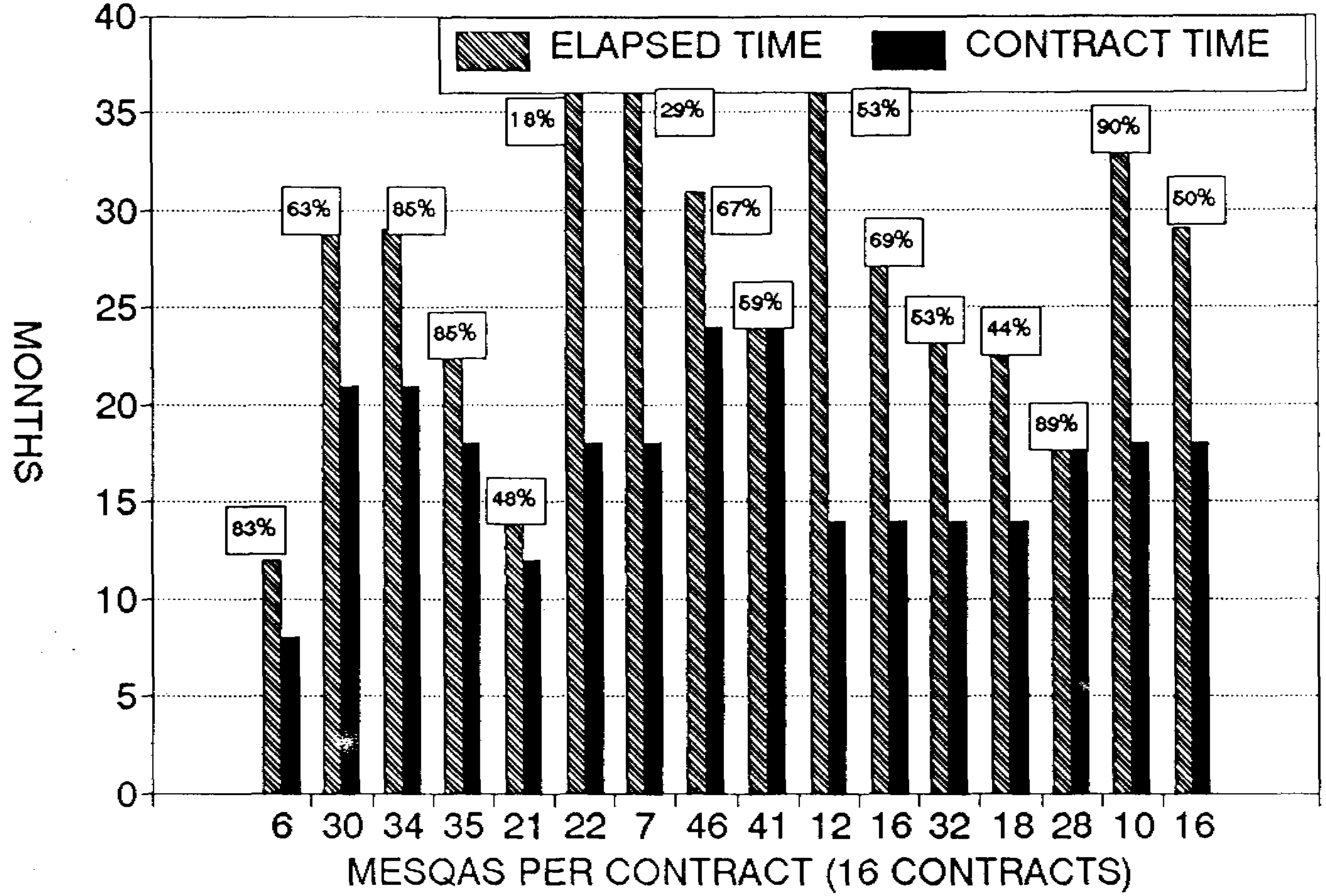

Figure 1 MESQA CONSTRUCTION, FEBRUARY 1994

CONTRACT vs ELAPSED TIME AND % COMPLETE

(T7-S2) 3.4

Table. 1 Monitoring Data showing benefits, before and after IIP project

Item	Before	After
- Wheat production in ardab/feddan	9.11	12.89
- Berseem production in ton/feddan	27.6	30.00
- New crops in % expected to be introduced	0	35
- Adequate water for good land preparation for summer season, 1993.	12	83
- Water adequacy for good production for summer season, 1993.	33	90
- Average L.E cost per hour for pumping water per feddan.	3.6	2.18
- Estimated total L.E. cost of pumping water/feddan for two cropping seasons	170.8	75.5
- Average L.E. cost to irrigate one feddan	7.84	4.39
- Time to irrigate one feddan of wheat, in hrs		
- Time to irrigate one feddan of maize, in hrs	5.5	2.1
- Time to irrigate one feddan of Berseem, in hrs.	5.2	2
	4.7	2.5

3. THE COMPLEXITY OF THE IRRIGATED AGRICULTURE SYSTEM IN EGYPT

If construction management is the problem, then efforts are to be directed toward solving this problem. In order to solve it, it is important understanding the real environment where the irrigation improvement projects need to be implemented.

The setting of the problem could be classified into parts:

The first is related to the complex nature of the irrigated agriculture system which could be described as follows:

- The cropping pattern is intensive where two or more crops per year per each unit of land are grown.

- The cropping pattern in most of the areas is the multi-cropping system where different crops are grown in one season (cotton, rice and maize in summer season, and wheat, sugarbeet and Egyptian clover (berseem) in winter season) or any other combination of field crops.

- The cropping pattern in some specific areas is characterized by both multi-cropping and inter-cropping system in one season (examples are Onion - Cotton, different types of vegetable crops) .

- Orchards and sugarcane are examples of perennial crops.

Secondly, in terms of the water rotation system, Table. 2 shows the current rotation schedule for the North of the Nile Delta Region.

Table 2. Current water rotation for the North of the Nile Delta Region.

Time Period	Ratio on-Days/off days	Total in days
Oct. 16 to closure	4/8	31
Closure		0
Closure to March 15	5/10	10
March 16 to May. 25	7/7	35
May 26 to Oct. 15	4/4	71
Total		147

The water rotation schedule system is different between the Nile Delta and The Valley due to different space and time allocations, the rules for different delivery schedules are according to the type of soil, cropping pattern, season and boundary conditions.

For example:

Two-turn rotation:	4 days on and 4 days off (Rice) 7 days on and 7 days off (Cotton)
Three-turn rotation:	5 days on and 10 days off (general crops/summer) 5 days on and 10 days off (general crops/winter). 7 days on and 14 days off (general crops/winter).

As an example for the irrigated agriculture system, Figure (2) shows the cropping pattern of single farmer in Abu Raya illustrating the mixture of crops, crop irrigation and the fragmentation of the farm into parcels and the parcels into fields. The numbers at the bottom are the irrigation dates, the numbers at the top are the planting and harvesting dates, the dark lines with numbers in parentheses are the irrigation gaps, and the large numbers in the space between crops are the turnaround times .

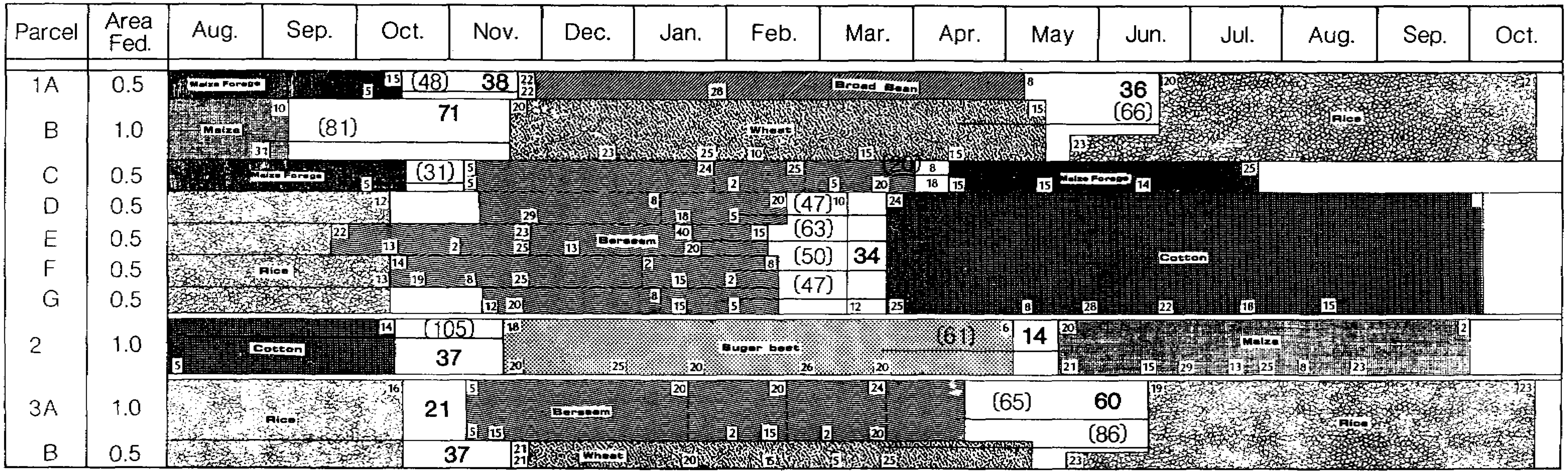

Figure 2: Cropping pattern of a single farmer in Abu Raya illustrating the mixture of crop irrigation, and the fragmentation of the farm into parcels and the parcels into fields. The numbers at the bottom are the irrigation dates, the numbers at the top are the planting and harvesting dates, the dark lines with numbers in parentheses are the irrigation gaps, and the large numbers in the space between crops are the turnaround times.

4. BUYING TIME IN WATER RESOURCES PROJECTS

The value of adapting a new construction management program parallel to and compatible with the intended water management program would give all the users, operators, managers, planners, and contractors the opportunity to buy time where all of them will have a mix of both direct as well as indirect benefits and approaching the potential savings of different resources, money, time, energy, water and human resources. In this case, only potential increase in crop production and the saved resources would increase the above listed parties' capacities and abilities to cover the costs of the new system as well as getting net benefits.

Another value of buying time is that it creates harmony between all parties involved in both water and construction management programs, it rebuilds trust between them and decreases the chances for confilt between any two and / or all the involved parties.

5. USERS' PARTICIPATION AS A PREREQUISITE FOR HIGH QUALITY CONTROL

Quality control of the implemented project should be performed along the developmental process of the project. In order to reach a high quality job, according to the engineering standards and specifications, users' participation is a prerequisite.

Since users participate before, during and after the turn over of the project, their ideas and suggestions through a feedback system are to correct the defaults in the implemented project and/or to modify the projects to be implemented in the future .

The sustainability of the new system depends on the satisfaction of the users and the operators of the system, and to a greater extent on the high quality control of the structure and its operating rules and roles .

6. OPERATIONAL PROBLEMS OF THE LAND LEVELING AND CONSTRUCTION MANAGEMENT

Interventions tested by EWUP to improve on-farm water management included precision land leveling, irrigation system design and management, irrigation scheduling, and crop management.

Precision land leveling activities were conducted at each of the Project sites. The operational problems of implementing the land leveling were many, but the most critical ones which could be of great value to utilize in a construction management program are accessibility, constraints, and timing. Many lessons could be learned from this project where constraints are either general / or local or specific to the site where the project is implemented.

6.1 Accessibility and Constraints:

The farmers' ability to perform PLL was constrained by the lack of roads for equipment movement, the small size of individual holdings, and the limited fallow time. Access to agricultural land was obstructed by washed-out inlets to mesqas and saqias, spoil piles left canal cleaning, trees, buildings, narrow roads, layout of on-farm channels in fields and wet fields.

Farm access during PLL intervention was by partial tracking across planted fields, driving through fallow fields, and fording irrigation ditches and drains after filling them with crop residues. Tractors and short implements could reach the field in most cases, but the longer field plane often could not be reached.

Field size was frequently one feddan or less. For such field sizes, maneuvering large field planes was difficult. To overcome this in Abyuha, the Project consolidation was promoted by the work of sociologists. In Beni Magdul and El-Hammami, the complex vegetable cropping patterns often resulted in fields of only 0.25 feddan. These fields were almost impossible to level.

6.2 Timing

Precision land leveling could only be conducted during the limited fallow periods, or turnaround times, between successive crops. These turnaround times had to be examined on both an individual field and on community-wide basis. The latter provided for better opportunity to move equipment from field to field. The turnaround time on individual fields was generally about one-half that for the community. Typical durations for turnaround are shown in Table (3). The field durations ranged from nearly 50 days to zero.

The latter occurred when berseem was sown prior to the harvest of either maize or rice. In Beni Magdul and El-Hammami the turnaround periods were usually less than 14 days, and more scattered throughout the year. This was the result of more intensive vegetable crop production in the area. Soil moisture conditions further limited the time period during which PLL activities could be performed. At Abu Raya, PLL was not done after rice cultivation because of very moist soil conditions.

Table 3. Turnaround Periods for Abu Raya and Abyuha:

Season	Site	Conversion	Average Per Field	Community
Winter to summer	Abu Raya	Berseem to Cotton	58	101
		Berseem to Rice	38	92
		Berseem to Maize	29	71
		Wheat to Rice	28	67
		Sugar beets to Rice	21	32
	Abyuha	Broad beans to Soybeans	33	72
		Broad beans to Cotton	14	28
		Wheat to Maize	34	53
		Berseem to Maize	26	82
Summer to winter	Abu Raya	Rice to Berseem	11	37
		Rice to Wheat	26	45
		Cotton to Wheat	31	61
		Cotton to sugar beets	29	62
		Maize to Berseem	24	71
	Abyuha	Maize to Broad beans	24	69
		Soybeans to Broad beans	47	92
		Soybeans to Broad beans	20	46
		Maize to Berseem	8	39
		Cotton to Wheat	47	63

7. THE NEED FOR CONTRACTORS TO BE TRAINED

In this case study, the research findings have been formulated when the irrigated agriculture production system was running under regulations set up by the government. Also, interventions were suggested to be implemented according to the tenders and bids system which gives priority to public construction companies over the private sector small enterprises - It does not mean that one of these public or private is always better than the other.

The large number of contracts whose completion exceeded two years more than the allotted time to finish the contracts explains that the understanding of these construction companies to the irrigated agriculture system practices is limited.

The crop calendar is one of the most important piece of information which describes the sequencing of crops, the irrigation scheduling of the different cropping patterns and the agronomic practices of land preparation , planting, growing, fertilization, weeding pest control and harvesting,... etc.

To implement projects in an irrigated agriculture system, a contractor should understand the system description and identify the period of time where he/she can go and implement his contract.

The contractors' understanding of the length of the contract for one year is 365 days per year but in reality this one year might end up earlier than 50 days distributed all over the year which are the period of time between the last irrigation of winter crop and the first irrigation of summer crops and vice versa plus some other limited times.

8. CONCLUSION

Modernization of the irrigated agriculture system in Egypt has become a necessity, so Egypt could fill its food gap as well as to satisfy future scarcity of water. In order to speed up the implementation process of the Irrigation Improvement Projects, Construction management program should be developed to be parallel and compatible with the irrigation management program. During the process of technology transfer contractors who execute the civil works should be trained so their awareness and capacities could be upgraded.

The value of time should be considered seriously when implementing water resources projects, because long delays in construction, the non-completion of the contracts and sometimes the poor quality of construction are most demoralizing to farmers and opposition to improvements could develop.

The need for construction management within the irrigated agriculture system necessitates users' participation in the process, training of contractors, and linking the applied research results to the implemented projects. All are important in order to maximize the benefit of the learned lessons during the implementation stage, obtention of high quality control of the project and increase in the performance of both hardware and the software of the new system.

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THE PERFORMANCE OF A LONG-TERM OPERATIONAL POLICY OF MULTI-UNIT RESERVOIR SYSTEMS UNDER DROUGHT CONDITIONS

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ABSTRACT

The paper presents an approach to derive long-term operational policy of a multi-unit reservoir system. Determination of the overall "near-optimal" operational scheme of a complex water resources system is based upon the decomposition of the system into single-reservoir subsystems and the subsequent successive optimization/simulation computational cycles performed on each reservoir. The interaction between reservoirs is maintained by computing the amount of water flowing from one reservoir to another. The optimization itself relies on the stochastic dynamic programming (SDP) based algorithm. Uncertainty is explicitly incorporated into the optimization procedure: the inflow to a reservoir described in a set of discrete classes with their respective independent or transition probabilities is considered as an additional state variable in the SDP-based optimization approach. Thus, the derived expectation oriented optimal policy tends to be really optimal as the operational period approaches infinity. Therefore it is particularly important to study the operational performance of the system under extreme drought conditions. This analysis can be carried out via simulation, by using individual drought years with arbitrarily set initial storage volumes or by analyzing observed or synthetical drought sequences within long inflow time series. In addition, both recommendations concerning project scheduling and decisions concerning the augmentation of existing reservoirs could be made. The applicability of the model has been tested within the national water master plan EAU2000 of Tunisia. The simulation results referring to increasing water demands for the different planning stages could be used to check the performance of individual system elements, as well as that of the whole interconnected system of 14 reservoirs.

RÉSUMÉ

Cette contribution présente une approche pour la déduction d'une politique opérationnelle à long terme à partir d'un système de réservoir multi-unitaire. La détermination d'un programme global opérationnel "quasi-optimal" d'un système de ressources en eau est basée sur la décomposition du système en sous-systèmes à réservoirs uniques et sur l'optimisation ou la simulation répétée que les cycles quantitatifs réalisent sur chaque réservoir. L'interaction entre les réservoirs est rendue par le calcul de la quantité d'eau s'écoulant d'un réservoir vers l'autre. L'optimisation elle-même repose sur l'algorithme basé sur la programmation stochastique et dynamique (SDP). Une incertitude est explicitement incorporée dans la procédure d'optimisation: le débit entrant d'un réservoir, décrit dans un ensemble de catégories discrètes avec leur probabilités indépendantes ou conditionnelles respectives, est considéré comme une variable de

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situation du système supplémentaire dans l'approche d'optimisation basée sur la SDP. La politique optimale dérivée orientée sur la prévision a donc tendance à être vraiment optimale quand la période opérationnelle tend vers l'infini. Il est donc particulièrement important d'étudier la performance opérationnelle du système dans des conditions d'extrême sécheresse. Cette analyse peut être effectuée par simulation, en utilisant des années de sécheresse distinctes avec un ensemble arbitraire de volumes initiaux de stockage ou en analysant les séquences de sécheresse observées ou synthétiques pendant des plusieurs séries de débit entrant de longue durée. En outre, cette contribution présente les recommandations concernant l'établissement du programme d'un projet ainsi que les décisions concernant l'augmentation des réservoirs existants. L'applicabilité du modèle a été testée dans le cadre du plan national de directeur pour l'eau de la Tunisie EAU2000. Les résultats de simulation se rapportant aux demandes accrues en eau pour les différentes étapes de développement pourront être utilisés pour vérifier la performance de chacun des éléments du système, ainsi que celle de la totalité du système de 14 réservoirs communicants.

1. INTRODUCTION

There are large spatial and temporal discrepancies between the availability of and the demand for water in arid and semi-arid regions. Multi-unit reservoir systems coupled with interbasin water transfer and conveyance structures play a very important role in redistribution of available water in both time and space. Long-term operational assessment of such complex systems is confronted with three major problems: i) high dimensionality regarding the number of state and decision variables needed to describe the system; ii) consideration of uncertainty which arises from extremely variable climatic conditions; and iii) highly complex water allocation composition due to the fact that multiple demands may be supplied from the common source (reservoir), or/and several reservoirs may provide water for a common user.

The method presented has been developed in order to provide a relatively simple approach in dealing with the three aforementioned obstacles. Determination of the overall operational scheme of a complex water resources system is based upon the decomposition of the system into single-reservoir subsystems and the subsequent successive optimization/simulation computational cycles performed on each reservoir. The interaction between reservoirs is maintained by computing the amount of water flowing from one reservoir to another. The optimization itself relies on the stochastic dynamic programming (SDP) based algorithm. Uncertainty is explicitly incorporated into the optimization procedure: the inflow to a reservoir represented by different classes with their respective probabilities or transitional probabilities is considered as an additional state variable in the SDP-based optimization procedure. A multiobjective decision problem that arises from the envisaged complex water allocation pattern is reduced to a single-objective optimization by aggregating individual requirements for water from each reservoir into a single composite demand. This simplification is supported by the arrangement of individual demands with respect to a predetermined priority order which is conformed with in the subsequent allocation of available releases from the reservoir.

Due to the stochastic nature of the inflow state variable, the derived expectation oriented operational policy tends to be optimal as the operational period approaches infinity. Furthermore, the "optimal performance" is pursued considering equally all probable outcomes of a hydrological state variable thus putting no specific emphasis on extreme events that is, droughts or floods. Therefore it is particularly important to check the operational performance of the system under extreme drought conditions. These analyses are carried out via simulation, by using individual drought years with arbitrarily set initial storage volumes or by analyzing observed or synthetical drought sequences within long inflow time series. The results obtained can provide valuable information on how much the derived policies are susceptible to severe conditions imposed upon the system. Clearly, any further refinement of an operational strategy should reflect the improvements in the system's performance regarding the conclusions drawn from such a sensitivity analysis. In addition, to test the performance of an expectation oriented policy under extreme dry conditions at the planning stage is understood to be a realistic setup to simulate real-time operational behaviour of the system considered.

The applicability of the SDP-based optimization model has been tested within the national water master plan EAU2000 of Tunisia. The system considered consists of 14 interconnected supply reservoirs and 2 major diversion weirs. While no shortages occurred for median or average hydrological years, regional shortages seem to be unavoidable during extended drought periods. The simulation results referring to increasing water demands for the different planning stages up to the year 2010 could be used to check the performance of individual system elements, as well as that of the whole interconnected system. As a byproduct, both recommendations concerning project scheduling and decisions concerning the augmentation of existing reservoirs could be made. The SDP-based planning tool can be extended to derive flexible policies for the on-line operation of the system.

2. METHODOLOGY

Stochastic dynamic programming (SDP) based operational assessment of a multi-unit reservoir system inevitably implies huge computer storage and processing time requirements. These are due to the inherent feature of dynamic programming (DP) that all states and decisions should be represented by discrete variables. As a direct consequence, the number of possible transitions of the system's states that have to be enumerated at one stage increases exponentially with the number of state variables. Computational difficulties further expand if the stochasticity of inflows to reservoirs is explicitly incorporated into the optimization procedure, thus introducing a number of additional state variables, describing river flows, into the SDP formulation.

Several strategies have been employed to overcome these dimensionality problems. They can generally be classified in four groups:

- In cases of systems consisting of only a few reservoirs a direct application of SDP could be opted for. This can be achieved by adopting a coarser discretization of state variables. It could, however, impede the accuracy of the analysis and could have negative effects on the convergence of the SDP-based procedure towards the optimal solution.
- Aggregation/disaggregation methods could be used to create a single composite reservoir representing the system or parts thereof. Consequently, optimization is carried out to derive the optimal operational strategy of the composite reservoir. Finally, the composite operational policy is disintegrated into individual reservoir operational rules.
- Different decomposition techniques have been proposed by many authors as powerful tools to tackle dimensionality problems. In general, a system is decomposed into smaller subsystems which allow use of more complex optimization methods than those applicable to complex systems. Apart from standard state and decision variables, a set of new variables should be introduced to describe the interaction between decomposed subsystems.

Decomposition techniques are essentially iterative procedures. Thus, the computations converge towards the stable, "near-optimal" or "local optimum", solution throughout repeated iterative runs. It should be noted that simulation is very often used in conjunction with optimization to provide the necessary flow of information from one iteration to the next.

- Combining two or more of the aforementioned approaches could also be found as one of the ways to overcome dimensionality problems in water resources management and planning.

The presented algorithm has been developed for long-term optimization of multiple-reservoir systems. It relies on decomposition of the system into individual, single-reservoir, subsystems. Each reservoir's operational policy is derived and assessed separately with respect to inflows from its local catchment, existing demands, and reservoir characteristics, while at the same time considering the inflows from upstream reservoirs and the presence of downstream reservoirs. The order in which the individual reservoirs are selected for optimization is not strictly fixed. It can partly be chosen freely, and partly it depends on the adopted approach (downstream, upstream, or up-and-downstream direction [*Milutin,* 1992]). The sequence of optimization may also reflect existing firm water allocation policies which can be treated as a type of constraint. This paper concentrates on the application of a sequential, "downstream moving" decomposition technique.

In essence, the algorithm is an iterative procedure. This implies that the final solution is obtained by repeating the principal computational cycle until the stable system return is observed. One computational run comprises a sequence of optimization and simulation procedures performed in turns on each reservoir of the system.

The core of the procedure is a variation of the explicit SDP algorithm given by *Loucks et al.* [1981]. In addition to the original approach which uses inflow transitional probability matrices to describe inflow stochasticity (i.e., the probability of inflow occurrence in subsequent time steps given the inflow at the present stage is known), the presented SDP optimization model allows the alternative consideration of independent inflow probabilities at each stage. The latter approach is thought to be efficient enough in cases of low correlation between river flows in subsequent time steps (e.g., weeks, months, seasons).

2.1. Decomposition

When considering optimization of a multiple-reservoir system operation as opposed to a case of a single reservoir, some principal differences have to be noticed:

- The inflow volume flowing into a reservoir might be enlarged by unutilized releases and spills of other reservoirs that are situated directly upstream of the reservoir in question.
- A particular demand may be supplied by more than one reservoir.
- Deficits in supply of the reservoir situated immediately downstream of the one considered are assumed to be one of its demand components. These deficits are composed as the sum of the individual demands that could not be met by the downstream reservoir. They can be obtained by simulation, following the "optimal" policy derived for the downstream reservoir.

The aforementioned issues are directly incorporated into the adopted decomposition scheme. Additional inflows coming from upstream reservoirs and deficits of downstream reservoirs are represented by supplementary variables that should provide links between

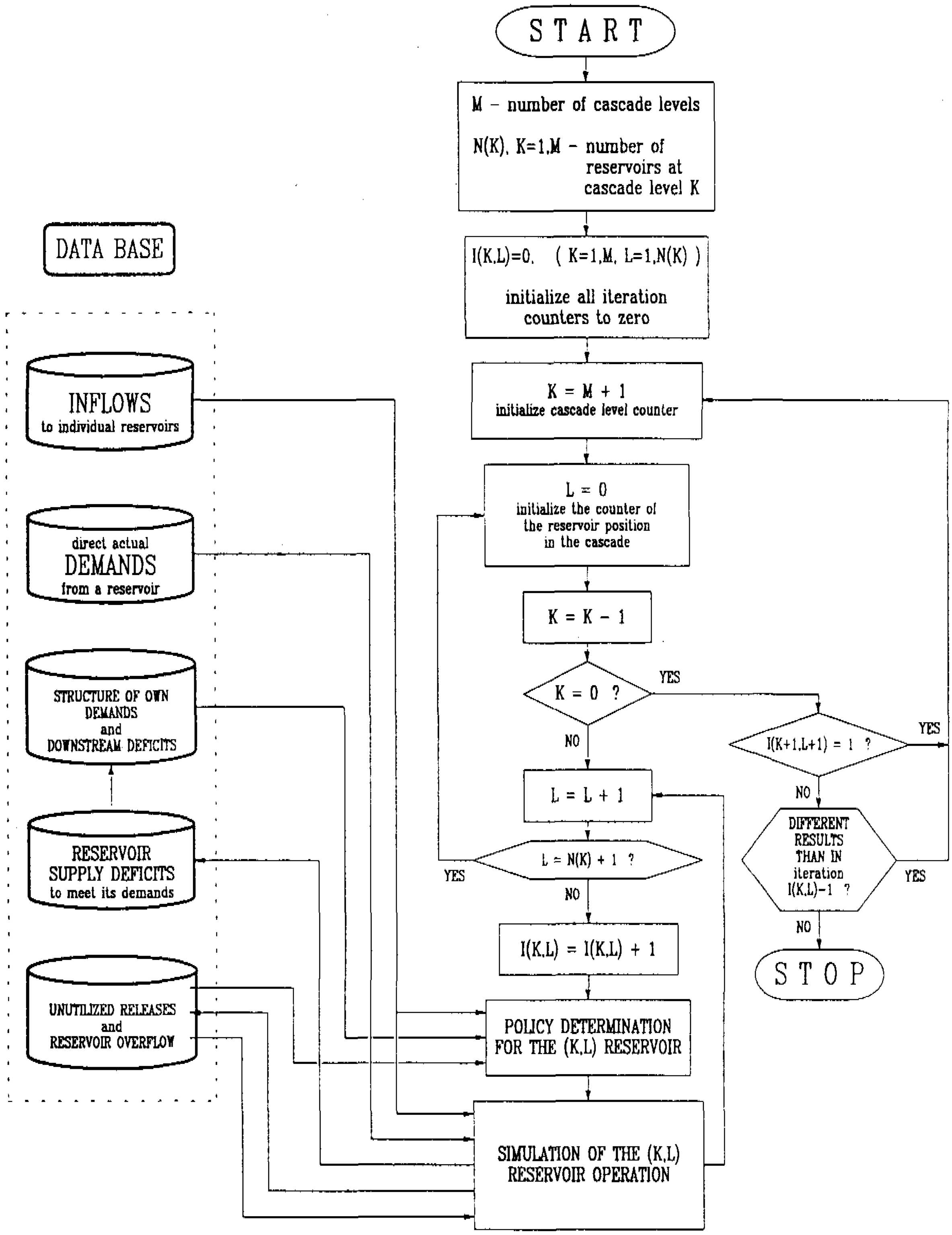
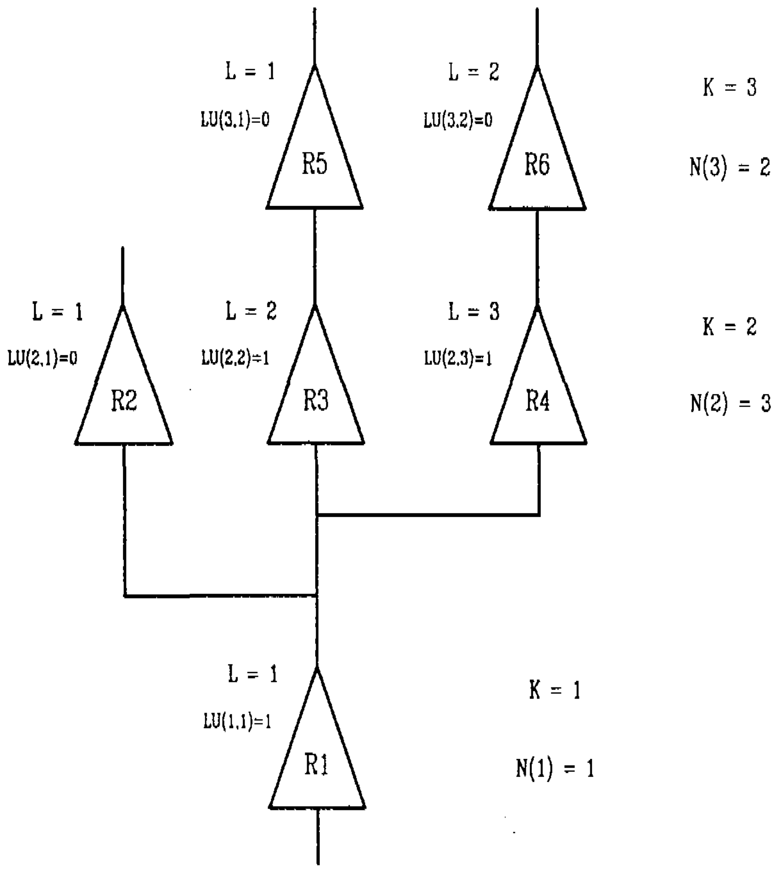


Figure 1. Sequential decomposition in optimization of a multiple reservoir system operation



LEGEND

R1, R2, R3, R4, R5, & R6	- reservoirs
M	- number of cascade levels
K	- cascade level counter
N(K), [K=1,M]	- number of reservoirs in cascade level K
L	- reservoir counter at a cascade level
I(K,L), [K=1,M; L=1,N(K)]	- iteration ordinal number of the (K,L) reservoir
LU(K,L), [K=1,M; L=1,N(K)]	- upstream reservoir identifier

Figure 1. (continued) The legend

reservoirs which are severed by decomposing the system into single reservoirs. Each reservoir operation is optimized with respect to the following factors:

- reservoir characteristics which comprise maximum and minimum storage capacity, the capacity of service outlets, and specific evaporation losses;
- incremental (own) inflows to a reservoir drained from the adjoining catchment area;
- additional inflows which are a contribution of reservoirs situated immediately upstream from the reservoir considered;
- the remainder of the respective demands that has remained to be covered after the contribution of all the reservoirs considered prior to the actual one has been estimated; and
- deficits in supply of the immediate downstream reservoir.

The procedure starts by optimizing the operation of the uppermost reservoir of the system, follows the downstream reservoir sequence, and ends up by optimizing the operation of the lowest lying reservoir in the catchment. Supply deficits obtained in the preceding iteration run are used to modify the actual demand of the reservoir being optimized. As these values are not known in the first iteration, they are assumed to be zero. Additional inflows to the reservoir are composed of non-consumptive releases from upstream reservoirs. They are estimated via simulation from the current iteration run. Iterations are repeated until no improvement of the system return in terms of water supply is registered. Figure 1 shows the flow chart of the sequential (downstream) decomposition algorithm.

2.2. Operation of a Single Reservoir

One iteration run comprises consideration of each reservoir of the system individually in the pre-specified order. The analysis of a single reservoir operation involves five distinctive phases:

- estimation of a demand for water placed upon the reservoir at a particular computational step;
- calculation of inflow volumes entering the reservoir over the period considered;
- optimization of reservoir operation based on SDP;
- assessment of the derived operational policy is carried out through simulation over the same inflow record used in optimization; and
- allocation of reservoir releases obtained by simulation and estimation of its supply deficits and remaining demands that are to be covered by other reservoirs.

In an SDP formulation of a water resources allocation problem, time periods are often considered as stages which are in this case set to monthly time steps. A year is decomposed into 12 monthly stages and the backwards DP recursion is applied.

Sets of characteristic storage and streamflow values are chosen so that the entire ranges of possible storage volumes and streamflows are represented. Hydrologic uncertainty of streamflows is explicitly taken into consideration. The model incorporates discrete probability distributions of monthly river flows (or alternatively the transitional probability matrices between subsequent monthly flows) into the optimization process. They describe the extent of the uncertainty of future occurrences of streamflows and the serial correlation of streamflows within a river basin.

The stored volume of water in the reservoir at the beginning of each stage (month) represents the explicit state of the system. The decision to be taken at each stage is the quantity of water to be released. That can be implicitly identified by specifying the targeted storage volume at the beginning of the next stage i.e., identifying the storage volume at the end of the time step considered. Stochasticity of inflows to the reservoir can be considered in two ways. One is to represent the flow process in a random state variable. Alternatively, to incorporate the uncertainty of inflows as a Markovian process rather than a random one, inflow to the reservoir can be considered as an implicit state variable. Therefore, an SDP formulation of a water resources allocation problem will have a two-dimensional state space consisting of the storage volume and the inflow to the reservoirs as state variables.

2.2.1. Objective criterion

Optimization itself pursues the optimal operation of a reservoir with respect to the objective function that minimizes the expected sum of squared supply deficits over an annual cycle. As an alternative objective, the sum of squared deviations of releases from demands can be opted for. Representative monthly demands are estimated as sums of individual (monthly) demand components associated with the reservoir. The developed algorithm allows two different approaches to be applied with respect to the definition of the demand used in optimization: a real and a 'fictive' demand composition. While the concept of a real demand is fairly self-explanatory, the 'fictive' demand approach obviously requires further clarification. This so-called 'hypothetical monthly demand concept' arises from the idea of providing the maximum challenge towards the utilization of the reservoir storage capacity, while the demand distribution remains unchanged. The concept is based on the hydrological regime of the corresponding basin, i.e. in areas with sub-humid to semi-arid climatic conditions the annual median inflow is used to represent the 'fictive' demand. The annual median inflow is redistributed with respect to the real (monthly) demand distribution within an annual cycle, i.e. the 'hypothetical' demand values are reflecting the distribution of monthly water requirements. This approach arises from the intention to create an overwhelmingly large demand. The 'hypothetical demand' is assumed to constitute a theoretical maximum demand a reservoir of unrestricted size, while having no losses whatsoever, would be able to fulfil without any shortage to occur. It is obvious that these prerequisites are not met by real-world reservoirs. Thus this hypothetical demand might be approximated, but never achieved. The record of available inflows is given as a time series of aggregated reservoir's incremental inflows and additional flows contributed by reservoirs situated immediately upstream. A set of constraints is imposed by defining minimum and maximum allowable reservoir storage and by restricting the release to take only positive values. And ultimately, Bellman's recursive equation is applied considering inflow to the reservoir as a random process i.e., inflow is represented by discrete probability distributions of monthly river flows.

Assessment of the derived operational policy is then carried out via simulation over the same inflow record used in optimization. However, these results describe only total monthly releases without any consideration paid to the fulfilment of individual demand components. Thus, in the subsequent step the allocation of available water among individual users is carried out with respect to the predefined priority order of water supply. Water allocation procedure results in reduced individual demands which are to be supplied by other reservoirs. In addition, average monthly supply deficits of the reservoir being optimized are estimated. These volumes are to be considered as an additional demand assigned to reservoirs situated immediately upstream. Finally, the records of non-consumptive releases from the reservoir are also estimated in order to be used as supplementary inflows to the existing downstream reservoir, if any.

3. THE CASE STUDY

The proposed decomposition algorithm was tested within a water resources master plan for Tunisia which was executed through the project EAU2000 [*Agrar-und Hydrotechnik*, 1993]. The main aim of the project was to derive and to assess feasible water resource management strategies for the country up to the year 2010. The presented methodology for a long-term assessment of multiple-reservoir systems operation was used to evaluate the contribution of major individual reservoirs and complex reservoir systems towards the national water balance throughout the envisaged development phases.

In this paper, the analyses regarding the role that the major 14-reservoir system could play in the provision of water over the national scale in the year 2010 are presented. The effectiveness of the derived SDP-based operational strategy is evaluated by comparing the respective expected long-term contribution towards the national balance to possible outcomes regarding the implementation of those policies on the operation of the system under dry hydrological conditions. In addition, the applications regarding project scheduling are illustrated on a 3-reservoir system for which the potential impact of inclusion of new reservoirs at different time horizons was assessed.

The system considered consists of 14, partially existing and partially in the planning stage, large reservoirs (El Kebir, El Moula, Sidi El Barrak, Sedjenane, Joumine, Zouitina, Ben Metir, Bou Heurtma, Kasseb, Mellegue, Tessa, Beja, Siliana, and Sidi Salem) and two diversion weirs (Mellita and El Aroussia) situated on the Medjerdah river, its tributaries, and in several adjacent river basins (Figure 2). These 14 supply reservoirs form a complex system with the joint objective to manage the available water resources of the basins. They form a network with both serial and parallel connections among reservoirs including a complex water allocation pattern consisting of a number of local and commonly shared supply targets.

El Kebir and El Moula reservoirs mainly provide water for the common local irrigation areas. In addition, the envisaged pumping schemes are planned to transfer a part of the water towards Sidi El Barrak. This reservoir supplies its local irrigation areas and, via a pipeline, contributes to the increase of incoming flow into Sedjenane reservoir. Sedjenane covers the demands for water of local irrigation areas and of remote municipal water users. It should be noted that those municipal areas are also, through an extensive interbasin water transfer system, supplied by Joumine, Sidi Salem and Siliana reservoirs.

Kasseb reservoir provides drinking water directly for the capital Tunis. This is its main purpose, largely due to its good water quality. Its non-consumptive releases of water can be used as additional inflows to Sidi Salem reservoir.

Ben Metir and Bou Heurtma reservoirs are in a cascade. The downstream one, Bou Heurtma, has basically its local irrigation demands to fulfil. Any remaining unused releases flow towards Sidi Salem. Ben Metir, with its water of high quality, first of all supplies Tunis with drinking water. Additional supply targets are three remote provincial residential areas. If there is any non-consumptive water left, it is released towards Bou Heurtma. In addition to this, Zouitina reservoir is planned only to increase the inflow to Bou Heurtma. The available resources from its own (Wadi Barbara) catchment are extended by diverting water from Mellita weir, situated in a neighbouring river basin.

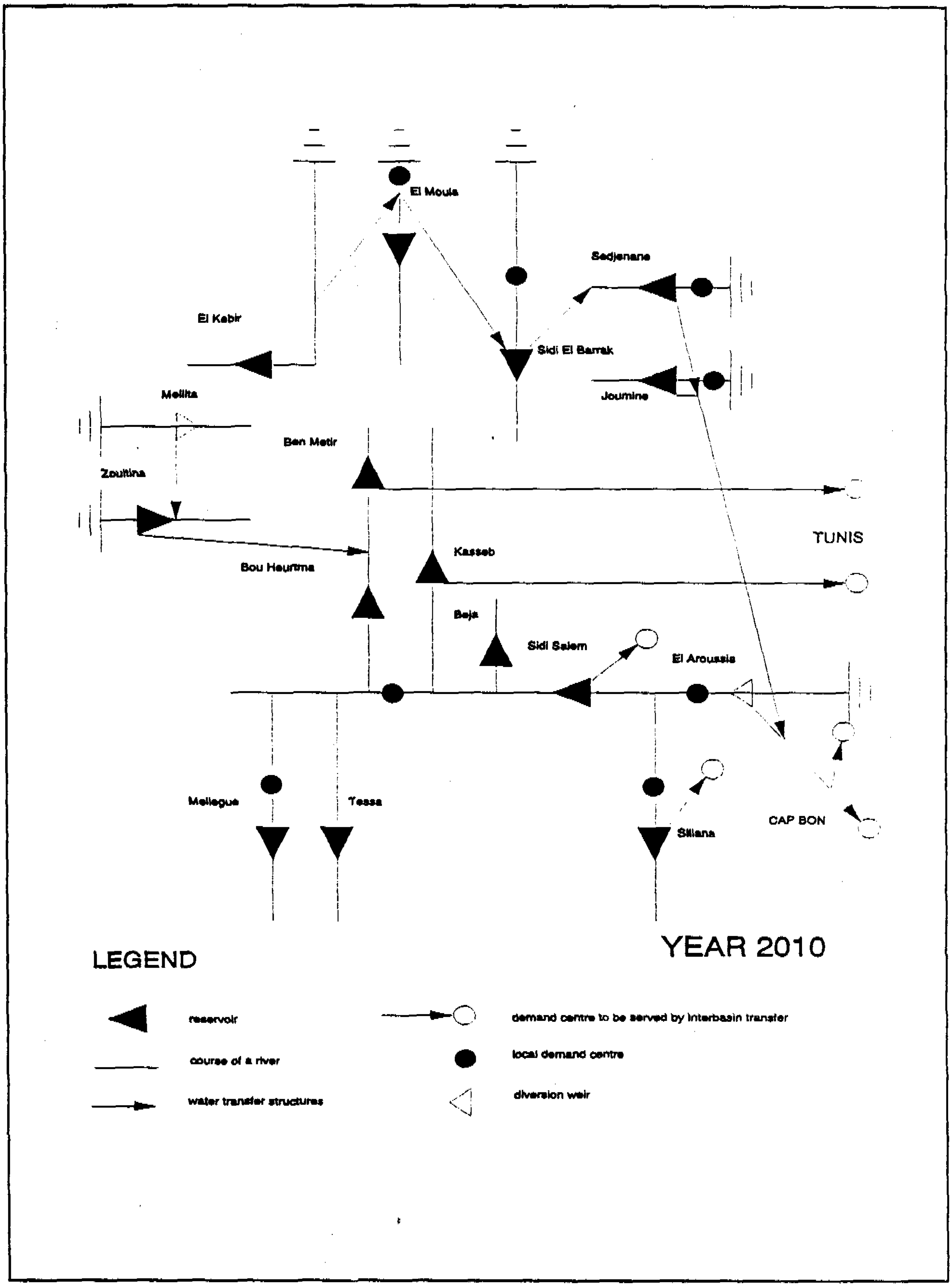


Figure 2. The complex reservoir system in Tunisia

Mellegue supplies its local irrigation demands and provides water for irrigation to areas that Bou Heurtma reservoir might have failed to supply. It is the third reservoir of the system from which unused downstream releases flow directly towards Sidi Salem to cover its deficits. Tessa provides additional supply for Bou Heurtma irrigation areas and releases water towards Sidi Salem. Beja reservoir however has, at this level of abstraction, the only purpose to regulate the flow from its own catchment area to Sidi Salem.

Sidi Salem is not only the most downstream but also the largest reservoir of the system. In addition to its own natural inflows, it utilizes non-consumptive releases from Bou Heurtma, Kasseb, Mellegue, Beja, and Tessa reservoirs. It allocates water for irrigation purposes, and further supplies remote urban and tourist centres with drinking water. Finally, Siliana provides water for local irrigation schemes, irrigation perimeters downstream of Sidi Salem, and remote irrigation and municipal demands.

4. ANALYSES AND RESULTS

This Chapter presents two different application aspects of the proposed decomposition algorithm for long-term operational assessment of large scale systems. In the first part, the analyses regarding the outcomes that the derived SDP-based operational strategy of a 14-reservoir system provides if applied under drought conditions. The second Section presents the application of the derived algorithm regarding problems of project scheduling. Namely, the operation of a 3-reservoir system (including one diversion weir) is evaluated at different phases of its development in order to define the most appropriate time for inclusion of new reservoirs into the subsystem.

4.1. Long-term Operational Policy Under Drought Conditions

The major contribution towards the water balance of Tunisia comes from a complex water transfer system which consists of 14 large reservoirs, 2 diversion weirs, and a number of channels and pipelines. This system provides water mainly for municipal supply, agricultural needs, large tourist centres along the Mediterranean coast, and industrial uses. Flood protection is also a major purpose of the system. In addition, most of the reservoirs have energy generation capacities installed. However, in this study the energy generation has not been considered.

Table 1. Anticipated monthly irrigation water demands in the year 2010 [$10^6 m^3$]

Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.
50.036	28.433	19.983	10.854	16.258	20.174	28.611	40.707	48.886	72.131	87.171	72.745

The anticipated monthly requirements for water in the year 2010 have been aggregated in 21 separate demand centres. Subsequently, each reservoir has been assigned a set of centres with their respective demand targets to provide water for. Inevitably, groups of reservoirs serving common demand centres could be indicated. The computations were carried out over a 44-year long inflow record (1946-1989) concurrently available for all reservoirs. The long-term operational policies for each reservoir were derived individually through iterative decomposition procedure based on SDP principles. A unique objective function that minimizes the annual sum of squared deficits of releases from a 'fictive' demand target was assumed for each reservoir.

Table 2. Monthly releases and the corresponding supply deficits of the entire system based upon the simulation following the derived long-term operational strategy

month	simulation over 1946-1989 (expected values)		dry year (1961) from simulation over 1946-1989		dry year (1961) initial storage: empty		dry year (1961) initial storage: half full	
	release [$10^6 m^3$]	supply deficit [$10^6 m^3$]	release [$10^6 m^3$]	supply deficit [$10^6 m^3$]	release [$10^6 m^3$]	supply deficit [$10^6 m^3$]	release [$10^6 m^3$]	supply deficit [$10^6 m^3$]
September	120.181	0.850	87.601	1.766	17.783	35.978	139.902	0.000
October	87.394	0.705	50.959	1.308	19.084	12.795	73.282	0.732
November	75.698	0.341	58.712	0.000	27.226	0.390	69.469	0.000
December	101.980	0.041	25.970	0.000	17.871	0.000	27.462	0.000
January	144.356	0.032	39.612	0.498	39.576	0.498	41.326	0.530
February	173.389	0.562	58.692	1.658	50.244	1.588	66.618	0.000
March	225.308	0.293	62.299	0.357	61.300	0.357	98.029	0.404
April	173.938	0.068	105.984	0.000	92.505	0.000	135.193	0.000
May	149.855	0.796	102.594	2.003	101.52	2.003	159.519	0.000
June	226.012	1.321	146.178	1.605	140.598	1.605	231.087	0.000
July	304.099	1.926	223.365	1.687	216.289	3.527	308.959	0.142
August	236.278	1.996	127.222	1.344	121.260	3.776	241.871	0.000
total	2018.488	8.931	1089.188	12.226	905.256	62.517	1592.717	1.808

Table 3. Initial storage volumes of the reservoirs as used in simulation runs

reservoir	initial storage for a dry year simulation [% of the active storage]			
	system is initially empty	system is initially half full	from 1946-1989 simulation	expected initial storage
El Kebir	0	50	60	20
El Moula	0	50	25	0
Sidi El Barrak	0	50	3	0
Sedjenane	0	50	100	100
Joumine	0	50	9	10
Ben Metir	0	50	10	10
Kasseb	0	50	23	10
Zouitina	0	50	33	0
Bou Heurtma	0	50	75	50
Mellegue	0	50	0	0
Tessa	0	50	0	0
Beja	0	50	0	0
Sidi Salem	0	50	0	0
Siliana	0	50	0	0

The distribution of actual water demand from the whole system is given in Table 1. The total anticipated annual demand amounts to $495.989 \times 10^6 m^3$, having 57% of the demand allocated in the period from June to September ($282.083 \times 10^6 m^3$).

The derived operational strategy was further used to derive the expected system return by simulating the system's operation over the whole investigated inflow record. Furthermore, the operation of the system is simulated over the characteristic dry year (1961) assuming three distinctive initial conditions:

- All reservoirs were set to be empty at the beginning of the simulation period.
- Initial storage volumes of all reservoirs were assumed to be at the level of 50% of the respective active capacities.
- Initial storage volumes were set to those derived for that particular dry year from the simulation over the whole inflow record.

Table 4. Monthly releases and the corresponding supply deficits of the entire system based upon the simulation over the dry year following the derived long-term operational strategy; total releases of Sidi Salem for different initial conditions

month	dry year (1961) initial storage empirically assumed		total releases from Sidi Salem reservoir; simulation over the dry year (1961) [$10^6 m^3$]			
	release [$10^6 m^3$]	supply deficit [$10^6 m^3$]	initial storage from 1946-1989 simulation	initial storage: empty	initial storage half full	initial storage empirically assumed
September	67.493	5.392	16.204	3.420	55.684	12.647
October	39.263	1.308	14.445	8.941	29.629	10.146
November	39.844	0.000	13.364	5.767	13.952	8.215
December	26.232	0.000	0.000	0.000	0.000	0.000
January	40.020	0.498	0.000	0.000	0.000	0.000
February	52.251	1.625	10.886	10.701	10.453	10.701
March	61.148	0.357	15.115	15.038	32.294	15.037
April	96.538	0.000	30.854	27.495	55.582	27.445
May	102.135	2.003	32.368	32.858	76.984	32.764
June	138.014	1.605	58.465	57.089	104.433	56.789
July	220.672	2.399	85.235	85.955	108.873	85.741
August	122.620	1.653	50.592	49.986	94.186	49.789
total	1006.230	16.840	327.528	297.250	582.070	309.274

The results from these analyses regarding total monthly releases from the system as well as the obtained supply deficits are presented in Table 2. It should be noted that total monthly release represents three aggregated components of water withdrawal from a reservoir: 1) release allocated for both local and remote consumptive uses; 2) withdrawals aimed to cover the needs of the immediate downstream reservoir; and 3) spillage. The values given as supply deficits were derived regarding only the consumptive release component.

It was concluded that neither of the three sets of initial capacities used (i.e., the system is assumed to be either initially empty, half full, or the initial storage can be assumed to be as that derived by simulation over the whole inflow record) could be taken as absolutely representative description of the system's initial state at the onset of the dry year (1961). Statistical analysis of the initial storage volumes of each reservoir obtained by simulation over the whole inflow record showed that the expected initial capacities could be identified as presented in Table 3. The subsequent simulation using the proposed initial storage states resulted in total monthly releases and supply deficits as presented in Table 4. Table 4 also contains information on total monthly releases from the biggest reservoir of the system, Sidi Salem, in all tested cases.

4.2. Project Scheduling

Through the period 1990-2010 the development of agricultural production in the North-Western region of Tunisia brings about a significant increase of irrigation water requirements. To illustrate the application of the SDP-based operational analysis of multiple-reservoir systems in project scheduling two particular irrigation areas in this part of the country are selected, namely the agricultural perimeters of Bou Heurtma and Mellegue reservoirs. These two reservoirs provide water for two large irrigation schemes: 1) the agricultural areas situated immediately downstream of Bou Heurtma are supplied by both reservoirs; and 2) in addition, Mellegue is providing water for irrigation in the local area. Therefore, irrigation water requirements are aggregated into two separate components, the so-called IBH and INE water demands representing the two irrigation schemes respectively (Figure 3). The anticipated increase of the demands for water from these two reservoirs is given in Table 5 [*Agrar-und Hydrotechnik,* 1993].

Table 5. Annual irrigation water demands at five development stages of the system

irrigation area	Anticipated annual demand for water [10^6m^3]				
	year 1991	year 1995	year 2000	year 2005	year 2010
IBH	57.00	70.00	112.90	142.00	151.30
INE	0.00	2.00	2.73	2.73	2.73

In order to improve water supply for irrigation in the future, the subsystem Wadi Barbara consisting of one reservoir (Zouitina) and one diversion weir (Mellita) is proposed to come into operation from the year 1995 onwards. Zouitina reservoir is anticipated to be in operation in 1995 whereas Mellita weir is planned to be built by the year 2000. The main purpose of this subsystem would be to regulate inflows in the local catchments and to transfer the water towards Bou Heurtma. Water from Mellita weir would be diverted towards Zouitina reservoir in monthly volumes up to $12\times10^6m^3$. The envisaged capacity of Zouitina-Bou Heurtma transfer would be $18.5\times10^6m^3$ per month. It should be noted that Ben Metir reservoir which is situated immediately upstream of Bou Heurtma is excluded from the analysis. This is warranted by the conclusion drawn from the assessment of the entire 14-reservoir system operation that the contribution of Ben Metir towards the increase of incoming flows to Bou Heurtma was insignificantly small with respect to the respective impacts of Barbara subsystem. Ben Metir cannot release significant amounts of water towards Bou Heurtma due to its own high demands imposed by the city of Tunis.

The analysis was carried out in three phases, each of which included the consideration of different system configuration throughout the proposed time horizons:

- The long-term operation of the Bou Heurtma-Mellegue subsystem (BH-ME) was derived and evaluated at development stages in the years 1991, 1995, 2000, 2005, and 2010.
- The Zouitina-Bou Heurtma-Mellegue subsystem (ZO-BH-ME) operation was assessed in 1995, 2000, 2005, and 2010.
- The operation of the complete Mellita-Zouitina-Bou Heurtma-Mellegue system (MA-ZO-BH-ME) was analyzed in 2000, 2005, and 2010.

The effects of siltation resulting in decrease of active storage of reservoirs were taken in consideration by estimating the expected dead storage volumes of reservoirs at each of the five development stages. SDP-based optimization algorithm was used to derive expectation oriented operational policies for each reservoir. The objective used in all cases was to minimize a sum of squared monthly supply deficits (real, not 'hypothetical' demand) over an annual cycle.

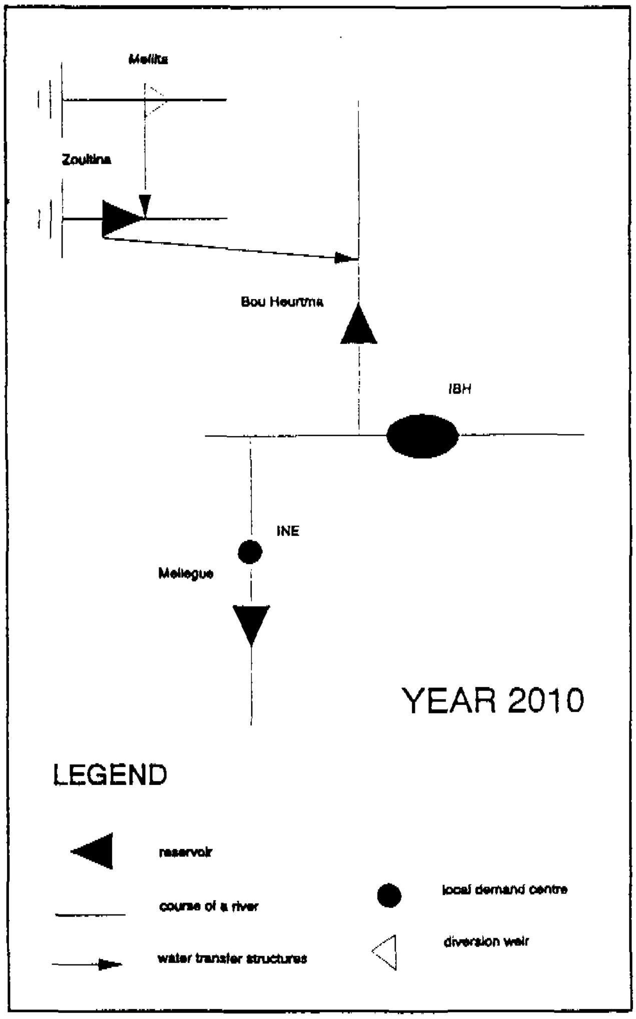


Figure 3. The subsystem scheme

Table 6. Expected annual water supply from the whole system and Bou Heurtma alone

system	provided by	Average annual supply [$10^6 m^3$]				
		1991	1995	2000	2005	2010
BH-ME	Bou Heurtma	53.68	62.51	75.45	78.62	78.15
	the whole system	56.85	69.50	111.43	136.09	137.98
ZO-BH-ME	Bou Heurtma	-	68.44	103.58	117.91	122.36
	the whole system	-	69.91	112.01	140.70	150.59
MA-ZO-BH-ME	Bou Heurtma	-	-	107.58	123.57	126.65
	the whole system	-	-	112.58	140.98	150.49

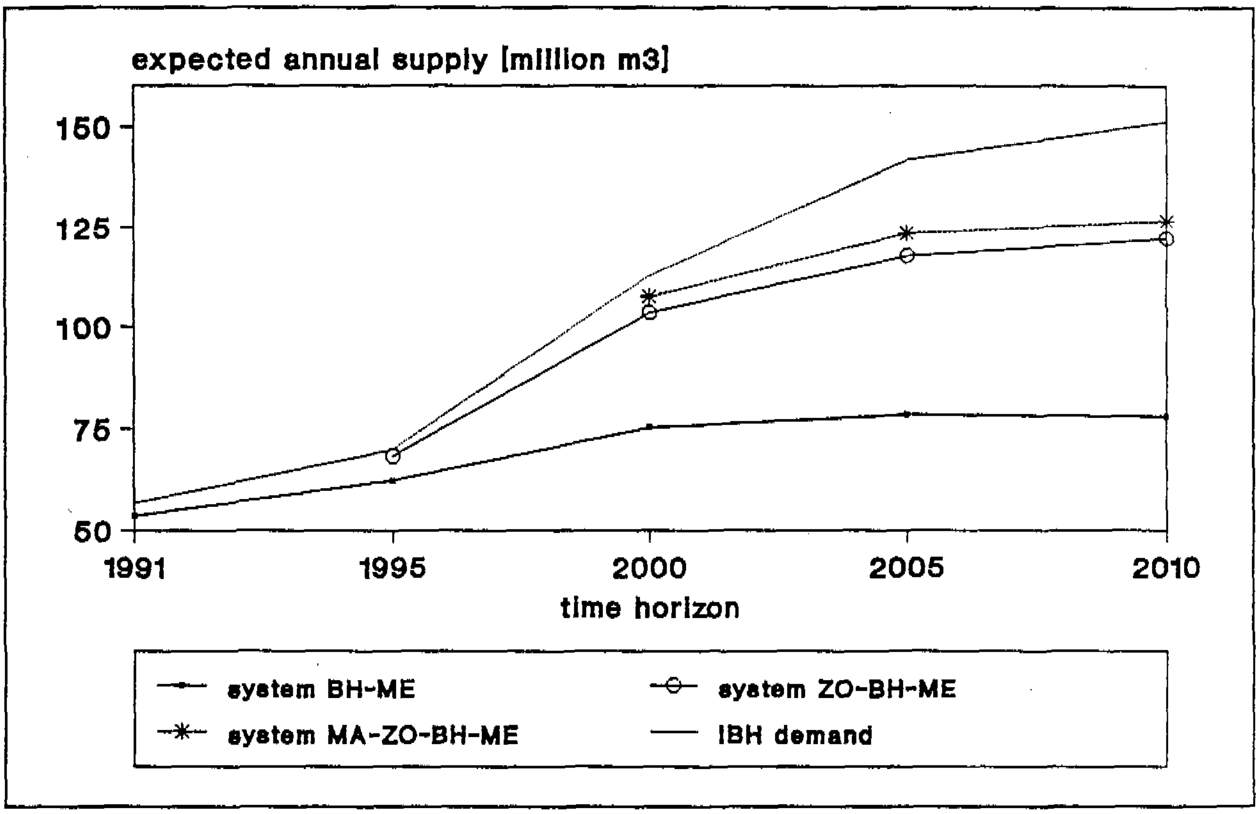


Figure 4. Expected annual irrigation water supply from Bou Heurtma

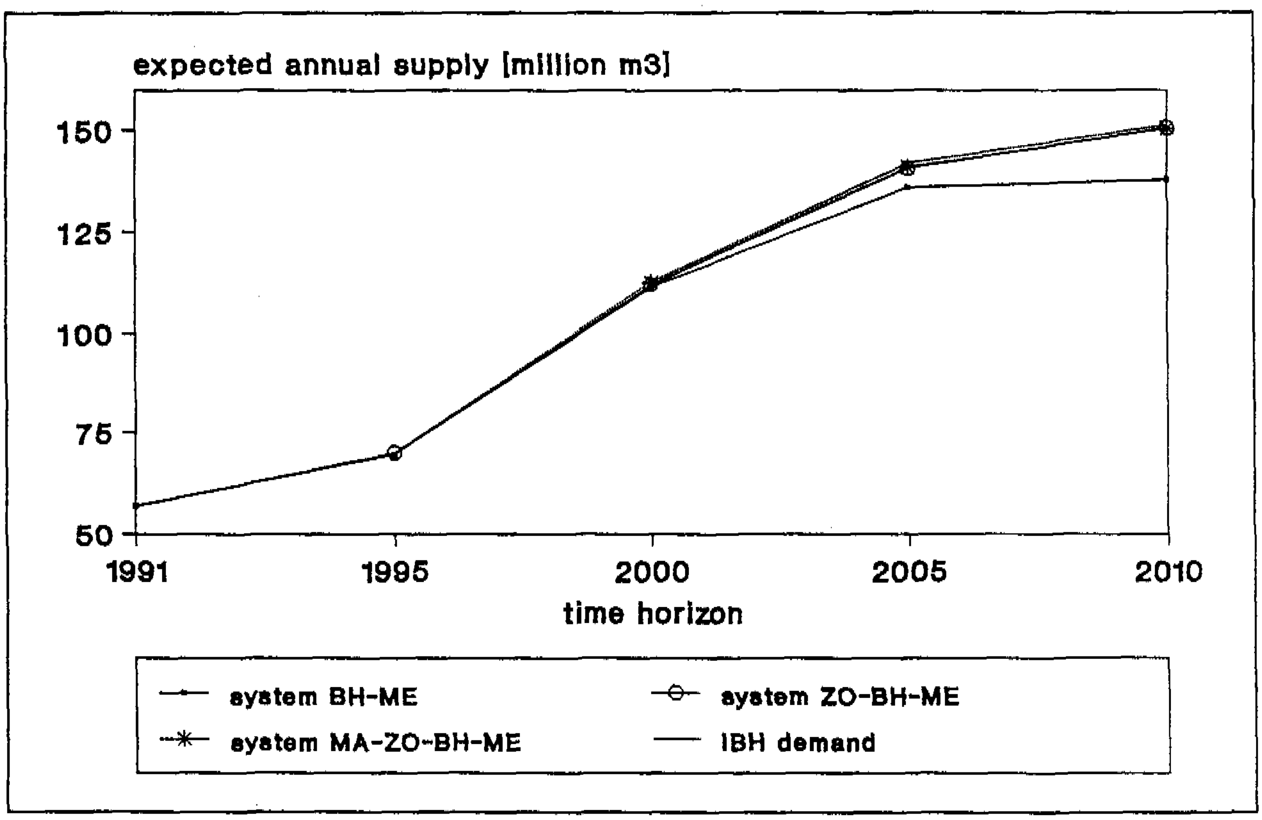


Figure 5. Expected annual irrigation water supply from the whole system

In the subsequent simulation over the same inflow record used in optimization the respective reservoir releases were derived. Finally, the releases were used to estimate the average annual volumes of water allocated to different demand centres. The results regarding the amount of water supplied for the IBH demand are summarized in Table 6 and graphically represented in Figures 4 and 5. As for the INE demand, it was observed that this relatively small volume could be easily provided by Mellegue reservoir in all examined cases.

5. CONCLUSIONS

The derived approach to assess long-term operation of multiple-reservoir systems and the respective computer model proved to be applicable to a number of water resources planning problems. The presented applications include a global operational assessment, sensitivity analysis of derived policies under drought conditions, and project scheduling. In addition, the sequential decomposition concept based on SDP was tested on cases of reservoir capacity expansion, impacts of siltation on system performance, and control problems of reservoir systems with purely recreational purposes. Thus, the approach showed an extensive applicability to reservoir systems operational analyses. Nonetheless, the advantages could be seen in its relatively simple and easy-to-understand method of system decomposition, the ability to accommodate inflow uncertainty and a fairly complex water allocation structure while pursuing the long-term optimal strategy with respect to the selected objective.

With regard to the presented analyses of the 14-reservoir system operation under drought conditions assuming the long-term operational strategy, the main points drawn from the results can be summarized in the following:

- Although the expected annual withdrawal from the system is as 4 times the demand some shortage still occurs. This is partly due to accuracy limitations imposed by discretization of storage and inflow variables. On the other hand, the adopted decomposition approach inevitably implies that the derived operational strategy is rather "near-optimal" than the global optimum of the system's performance. Furthermore, high temporal and spatial variability of river flows certainly causes supply shortage in some areas. Nevertheless, this relatively low shortage was expected to be acceptable on the long run.
- As for the assessment of the system's operation under drought conditions, it can be concluded that simulation runs carried out under different assumptions regarding initial system states can provide considerably broader insight into the effectiveness of SDP-based operational policies. For instance, the assumption that the system could have the half of its capacity available at the beginning of a dry year showed to be too optimistic. This statement is warranted by unexpectedly high output and almost insignificant supply deficit as well as by the outcome of the analysis of initial states at the beginning of each year obtained by simulation over the whole available inflow record. On the other hand, the pessimistic approach (i.e., the system being completely exhausted at the onset of a drought) proved to be closer to reality, at least as far as the total annual withdrawal from the system is considered. The respective huge supply deficits are mainly due to shortage observed in two initial months when no storage is available to compensate for low incoming flows. However, the water stored in the system at the onset of the dry year (1961) as given if it was excerpted from the simulation over the whole inflow record, or when the initial state of the system was empirically assumed was sufficient to cover the demands in the first two months substantially.
- In all cases except if the system is half full at the beginning of the dry year the supply deficit in the period from the fourth month (December) onward was observed to be almost identical.

The only significant difference was registered in July and August. This leads to two general conclusions: 1) due to very low inflows the initial reserve in the system was sufficient to augment the supply only in the first three months; and 2) July and August are the two critical months regarding water shortage. Thus, further analysis should be aimed at defining such an operating strategy which would mitigate supply deficits in these months.

As for the analysis regarding project scheduling the results reveal that the inclusion of the Wadi Barbara subsystem (i.e., Zouitina reservoir in 1995 and Mellita weir in 2000) would significantly improve the supply for the IBH demand. However, it is clearly noticeable (Table 6) that the contribution of Mellita weir towards the improvement of irrigation water supply in this area is almost insignificant. Thus, the inclusion of this structure into the subsystem under the anticipated conditions regarding irrigation supply could be ruled out.

With respect to the role of Mellegue reservoir, it can be seen that the available quantities of water are mainly provided by Bou Heurtma and the adjoining Barbara subsystem. This is due to the extensive siltation rate of Mellegue catchment. By the year 2010, the dead storage of Mellegue reservoir is predicted to expand from $31 \times 10^6 m^3$ in 1991 to $84.6 \times 10^6 m^3$, thus reducing its active storage by 60% (from $89 \times 10^6 m^3$ to $35.4 \times 10^6 m^3$). As a comparison, the active storage of Bou Heurtma was anticipated to decline in the same period by only 11% (from $102.5 \times 10^6 m^3$ in 1991 to $90.7 \times 10^6 m^3$ in 2010). Consequently, the amount of water that Mellegue reservoir would be able to supply towards meeting the IBH demand decreases significantly in time while it could still provide sufficient supply for the anticipated, but relatively modest, INE irrigation water requirements.

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