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This report describes a systematic framework for the analysis of long run investment needs in the water resources of the Sahel-Sudan region. The work was conducted as part of a larger project resulting from a contract between the United States Agency for International Development and the Massachusetts Institute of Technology's Center for Policy Alternatives.

Part I of the report summarizes key literature and data on the region's water resources. Part II includes descriptions and illustrative applications of two systems models that provide a framework for long run water resources planning in the region. Part III contains a detailed description of a proposed planning process appropriate for the long term planning of the region's water resources.

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A FRAMEWORK FOR EVALUATING LONG-TERM STRATEGIES FOR THE DEVELOPMENT OF THE SAHEL-SUDAN REGION

Annex 8

AN APPROACH TO WATER RESOURCE PLANNING

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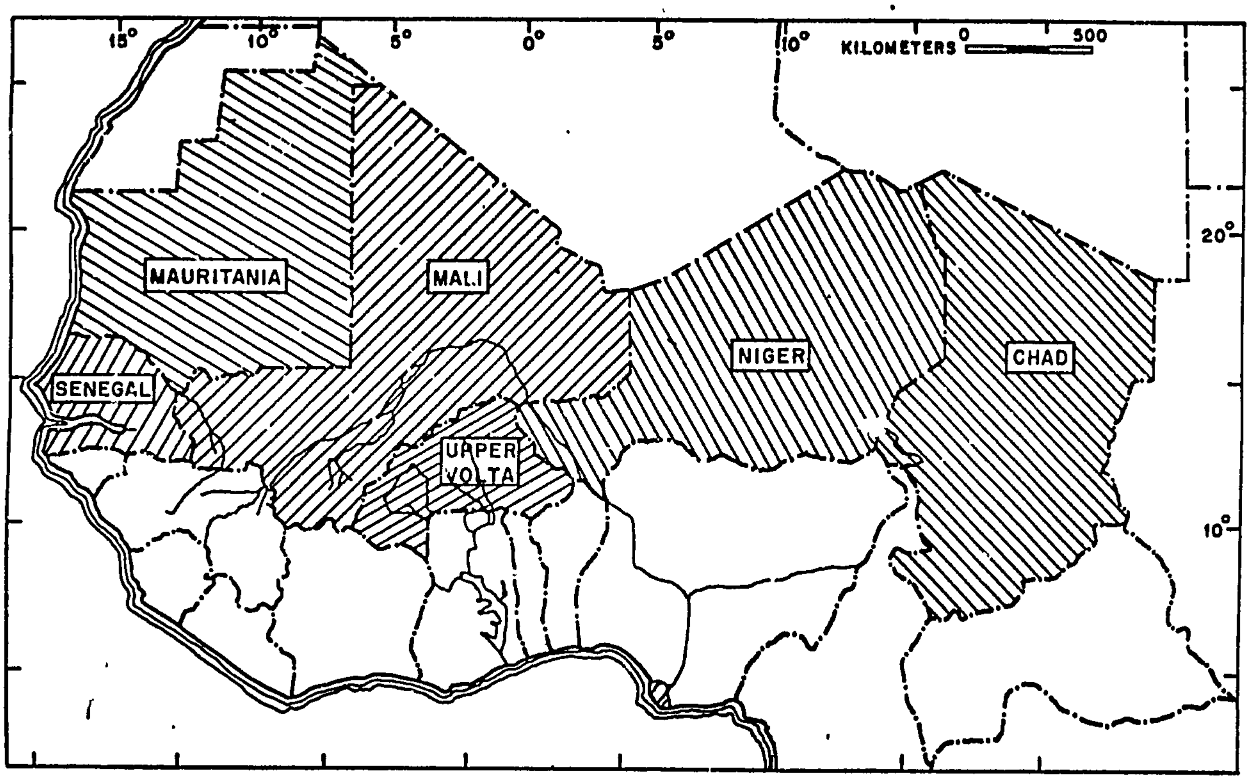
15. Abstract



Volume 1. Summary Report: Project Objectives, Methodologies and Major Findings

Volume 2. ISYALAPS, A Framework for Agricultural Development Planning

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The contents of this report reflect the views of the Sahel-Sudan Project at the Massachusetts Institute of Technology and do not necessarily reflect the official views or policy of the Agency for International Development.

FOREWORD

This report results from a one-year effort by a multidisciplinary team of analysts to establish a framework for evaluating long-term development strategies for the African Sahel-Sudan area.

By June 1973 it had become evident that the suffering caused by the drought was the most severe the area had experienced in the last half century. A meeting of donor organizations and U.N. agencies, called by the U.N., was held in Geneva to discuss the problem. It was clear that, while the area required immediate assistance to meet the problems of drought relief, there was also need for long-range assistance if the region were to become self-sustaining and begin an era of positive economic development and widespread improvement in the quality of life of its people. The U.S. delegation offered to undertake the first steps necessary to "identify the methodology, the data requirements and the possible alternative lines of inquiry from physical, economic, social and cultural points of view" on which to base "a comprehensive examination of technical problems and the major alternative development possibilities" for the region.* The United States Agency for International Development (A.I.D.) offered to take responsibility for this task and determined that it should enlist the assistance of the academic community in carrying out the work. A.I.D. then approached M.I.T., and a study effort was formally initiated with the signing of a contract covering the period September 1, 1973, through August 31, 1974. This contract was subsequently extended to January 1, 1975.

The goal of the U.S. effort is to develop a methodology for evaluating long-term development strategies for the Sahel-Sudan region. The

* Final Report on the Meeting of the Sudano-Sahelian Mid- and Long-Term Programme 28-29 June, 1973, Geneva. Special Sahelian Office, United Nations, New York. Statement by Donald S. Brown.

specific focus of the M.I.T. study has been on the development of an effective framework within which to appraise specific projects and programs. The term framework, in this context, refers to the accumulation, development, organization, integration, and analytical evaluation of information on the natural resources, economic resources, and human resources, including the social and political institutions, of the region. The framework is constructed in such a way that alternative strategies for the region can be identified and evaluated, in terms of both their requirements and their impacts, intended and unintended. The M.I.T. study has not been oriented toward detailed sector studies, prefeasibility studies, or project studies. Nevertheless, in the process of developing a methodology we have examined many kinds of information and a number of specific projects and have identified areas requiring further research to fill information gaps that impede long-range planning and evaluation of specific development proposals.

It is hoped that this framework will assist decision-makers in the Sahel-Sudan countries and in donor organizations in arriving at informed judgments concerning strategies for the long-term (20 to 25 years) social and economic development of the region.

The study was conducted under the direction of the M.I.T. Center for Policy Alternatives and was carried out by a multidisciplinary group. The Summary Report and the volume on agricultural development planning have drawn upon a number of working studies on specialized aspects of the problem prepared by the staff, i.e. (1) Economic Considerations for Long-Term Development, (2) Health, Nutrition, and Population, (3) Industrial and Urban Development, (4) Socio-Political Factors in Ecological Reconstruction, (5) A Systems Analysis of Pastoralism in the West African Sahel, (6) Technology, Education, and Institutional Development, (7) The Role of Transportation, (8) An Approach to Water Resource Planning, (9) Energy and Mineral Resources, and (10) Listing of Project Library Holdings and Organizations Contacted. The basic elements of these studies have been drawn together in the two volumes of the final report.

In addition to M.I.T. personnel, individuals from a number of other organizations participated in the effort. Participants from the University of Arizona, in particular, made major contributions; they had primary responsibility for developing the analysis of the agricultural sector strategy. Professor John Paden of Northwestern University was a major contributor to the work on socio-political factors. Members of the Société d'Etudes pour le Développement Economique et Social (S.E.D.E.S.) in Paris provided valuable insights into various aspects of the Sahel-Sudan area. Several members of the Centre de Recherches en Développement Economique (C.R.D.E.) in Montreal developed sections on monetary policy, urbanization, and relationships between Niger and Nigeria. A list of individuals who participated in the study is included in Volume 1 of this report.

Numerous other individuals acted as consultants to the project, provided advice as the study progressed, and reviewed draft material for the reports. Help and advice were given by officials of the governments of the Sahel-Sudan countries, the Comité Permanent Inter-Etats de Lutte Contre la Sechêresse dans le Sahel (C.I.L.S.S.), members of United Nation organizations, members of the International Bank for Reconstruction and Dévelopment, and, especially, officials of the Secretariat d'Etat and various socio-economic and technical study groups in France. Finally, representatives of A.I.D. arranged meetings in Africa and reviewed the the progress of the study. All this assistance is gratefully acknowledged.

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INTRODUCTION

This report describes a systematic framework for the analysis of long run investment needs in the water resources of the Sahel-Sudan region. The work was conducted as part of a larger project resulting from a contract between the United States Agency for International Development and the Massachusetts Institute of Technology's Center for Policy Alternatives.

Part I of the report summarizes key literature and data on the region's water resources. Part II includes descriptions and illustrative applications of two systems models that provide a framework for long run water resources planning in the region. Part III contains a detailed description of a proposed planning process appropriate for the long term planning of the region's water resources.

Our work from September, 1973 through August, 1974 was intended as the first stage of a continuing project to apply systems techniques to the long-run planning of the water resources of the Sahel-Sudan region. Although the project was not continued, and despite the limited manpower and budget that we were able to bring to our work in the water sector, we feel that the framework presented here is a correct one for the long-run planning of water resources in the Sahel-Sudan region, and we hope that some of the region's countries and donor agencies can use our work as an effective tool to program aid in the water sector.

During the first months of the project, personnel in the water sector familiarized themselves with the region's water systems, problems, and potentials. This effort included an extended visit to the region and to Paris to discuss problems with knowledgeable persons, and to collect data. Some of the information and literature collected is presented in Part I, both as an introduction to the water resources of the region for the reader not familiar with them, and as a description of some of the broad data used in the models presented in Part II.

Part II contains the proposed framework for long run planning of water resources in the Sahel-Sudan region. Following our initial data-gathering efforts, we made a careful assessment of the role that systems models might play in long range planning of water resources in the region.

This assessment convinced us that, although there are various possibilities for detailed modelling of particular systems in the region that will doubtless repay future efforts, these did not compare in importance for long range planning to the construction of a suitable framework for planning of the kind presented here. This framework is an essential first step in the rational allocation of aid resources to the development and management of the water sector.
Astonishingly enough, given the key role of water in the region, there has never been an overall framework for the analysis of water resource development in the region.

The two models in Part II represent our attempt to create the central elements in such a framework. Applied for selected sample areas in the region, the models permit us to forecast water requirements using a wide range of assumptions about the future, and to estimate the optimal sources and costs of supply that meet those requirements. The models can also be used to estimate the best allocation of limited aid funds within the water sector, and to study the consistency of presently proposed projects with overall demand forecasts.
The models were designed to be used in conjunction with the basic approach of the overall project of which our work was a part; to make

projections of main development alternatives and to investigate the investment requirements for them. Our models are designed to provide the framework for accomplishing this in the water sector within an interdisciplinary planning process. The water demands used in the illustrative applications of the models, for example, were developed on the basis of sectoral projections made by our colleagues in the project. The models are also designed, however, to be used for water sector planning alone, should there not be an overall planning framework such as the one developed in this project.

The requirements model is a FORTRAN IV model that takes projections of economic and demographic series as inputs, allocates them to water planning regions, and applies water use coefficients. The supply model is a mixed-integer mathematical programming model that can be used either to find least cost sources of supply or to maximize net benefits from investments in the water sector. (These models can be used to obtain a basic idea of the region's future supply/demand balance; then, the supply model can be run in a net benefit maximizing format with budget constraints to examine optimal allocations of aid resources to the water sector.) These models, used with extensive sensitivity analysis, provide information on where additional data is needed, where detailed source development studies should start, what are approximate development costs to meet objectives, and what sources and demands should be developed given certain market assumptions.

Because one of the problems of the region is the development of a planning format with which to use systems techniques in any sector, we have described in Part III a planning process appropriate to the use of our models for long run water resources planning.

PART I: HYDROLOGY AND WATER SYSTEMS OF THE SAHEL-SUDAN REGION

1.1 Introduction

Part I discusses water conditions and data in the Sahel-Sudan region. The six countries studied and their populations are:

Country	Area (Km.^{2})	Population (millions)
Chad	1,284,000	3.8 (mid 1971)
Mali	1,240,000	5.1 (mid 1970)
Mauritania	1,030,300	1.2 (mid 1970)
Niger	1,[illegible]65,900	3.7 (mid 1970)
Senegal	1[illegible]6,000	3.9 (mid 1970)
Upper Volta	273,900	5.1 (mid 1970)

Topics covered are: climatic factors; surface water; ground water; water resource use in the area; problems accompanying water resource development; and some effects of the present drought.

1.2 Climatic Factors

1.2.1 Prevailling winds in area: Africa straddles the equator where there is a low pressure belt, characterized by ascending air masses. North and south of the equator lie tropical high pressure belts characterized by descending and divergent air masses. Winds from the high pressure areas, which converge in the equatorial low-pressure belt are generally easterly, (i. e, northeast and southeast trade winds) due to the earth's rotation. The variation in African climate results from the north-south shift of these zones as the earth revolves around the sun.

During northern winter, when the zones shift south, all of Africa north of the equator is under the influence of the northeast trade winds. West Africa is subjected during this period to the harmattan, a dry wind coming from the Sahara Desert.

The zones shift north in northern summer. Since the convergence zone lies generally north of the equator, southwestern North Africa is subjected during this period to the hot, moist air from the southwest (the winds are diverted northeast as they cross the equator). Heavy rainfall occurs along the coastal areas as the moist air encounters the tropical dry air. The tropical dry air plunges beneath the moist air and lifts it up. The amount and duration of precipitation decrease northward.

1.2.2 Precipitation: There are two types of rainfall in the region. Line squalls, during July and August, generally occur north of 11^{o} N (just south of the southern border of Senegal). They are characterized by a short preliminary period of average intensity rainfall followed by from 5-20 minutes of very high intensity rainfall (50 - 100 mm/hr) when the bulk of the rain falls. Light rain ensues. Monsoon rains occur in the southern portions of West Africa (not in the Sahel-Sudan region) during a 5-6 month wet season which is interrupted by a dry spell in July and August. They are generally characterized by several periods of low intensity rain lasting several hours. Most of the area's rainfall comes from monsoons, with line squalls occurring mainly at the beginning and end of the wet season. Average monthly rainfall is shown in Figure 1.2-1.

The Sahel-Sudan region thus receives its rainfall in short intense storms (single peaked) during July and August, by contrast to southern West Africa's long monsoon season (double peaked) and a dry period in July and August. In both areas, rainfall for more than a few hours or more than one period within 24 hours is unusual. By analyzing daily rainfall amounts for areas exceeding 200 mm of rain, Ledger (1964) has found that the increase in rainfall southward is due to an increase in the number rather than the size of storms. Extreme variability of annual rainfall amounts is also characteristic of the region. Dresch (1973) reports that the "shorter the rainy season, the more irregular it (the rainfall) is in space, in time over the year, and from one year to another. The rains may start earlier or later, and may last a longer or a shorter time."

The area is thus also subjected to wet and dry years. Dry periods occurred from 1910 to 1914 (accompanied by famine) and from 1941 to 1942. The period from 1951 to 1960 was wetter than usual; from 1968 to the present, drier than usual. Table 1.2-1 shows a comparison between 1972 annual rainfall and average annual rainfall at several stations.

On a larger time scale, Northern and Western Africa have been subjected to extremely wet and dry periods. During the dry periods Sahara winds pushed dunes 200-300 km further south than they are today. In the most recent wet era, from 2300 to 6000 years ago, the Sahara had extensive lakes. The area's rainfall during the European Middle Ages was approximately double what it is today.

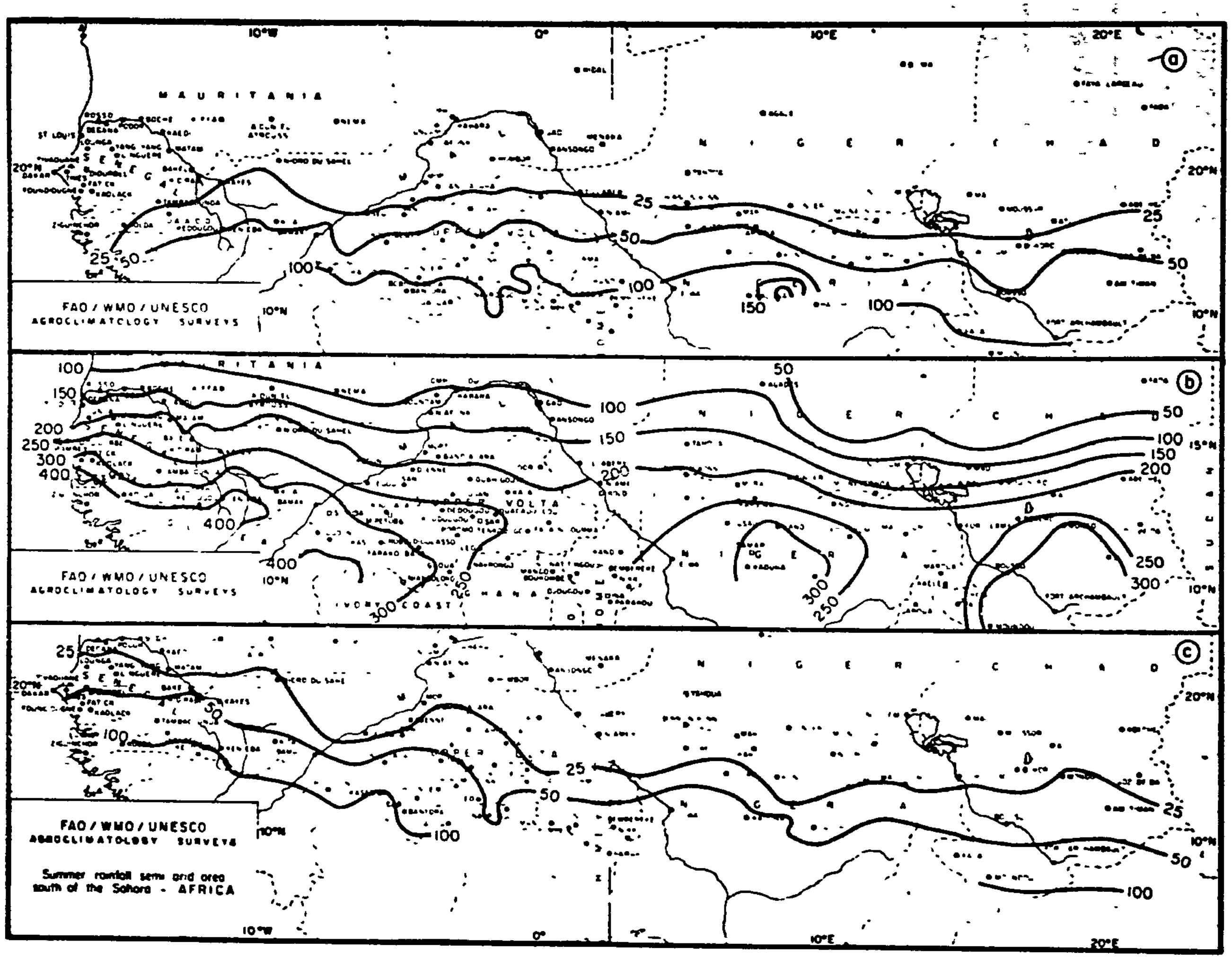


FIGURE 1.2-1 Average monthly rainfall (mm) in May (a), August (b), and October (c).

Source: Cocheme and Franquin 1967.

TABLE 1. 2-1

Comparison Between Average Annual
and 1972 Rainfall

	KAEDI Mauritania		NIAMEY Niger		FORT LAMY Chad		KAOLACK Senegal	
	1972	Average	1972	Average	1972	Average	1972	Average
	mm		mm		mm		mm	
April	—	2	11	3	—	5	—	—
May	—	3	16	24	34	36	—	8
June	27	29	55	79	96	65	82	61
July	9	87	102	101	187	156	16	160
August	60	166	82	206	137	257	164	295
September	22	95	57	100	85	104	188	201
October	10	20	16	21	60	23	40	64
November	—	4	—	—	6	1	—	4
TOTAL	128	416	343	534	599	647	400	793
	in days		in days		in days		in days	
Vegetation Period	—	40	15	75	65	80	55	100

Having statistically analyzed rainfall data from Dakar, Senegal for the years 1889-1972, Landsberg (1973) states, "The data there show rather convincingly that there has been no major change in climate, only fluctuations that are to be expected by the geographic position of the Sahel and the random variations one can expect in precipitation patterns. Major droughts are a part of the climate there." Douhin, a French hydrologist who has also studied the area, also supports this view (Cook 1973). The fluctuations (which may be cyclical) could be more or less related to astrophysical situations; forces beyond the earth, such as sunspots; lunar effects; and evolution of the earth's magnetism.

Rainfall data and statistical analysis are available in the reports of Landsberg (1973) and Cocheme and Franquin (1968). However, Roche (1973) notes that statistical data are insufficient to determine really if the droughts are cyclical and/or if the climate is generally getting drier. In some areas, rainfall data is available since the beginning of the 20th century. However, the density of recording rain gauges is low.

1.2.3. Temperature distribution: Average temperatures vary from about 30°C in spring and summer, to about 24°C in fall and winter.

1.2.4. Evapotranspiration: A good indicator of the interaction of climatic effects is an area's potential evapotranspiration, defined as the evaporation and transpiration loss that would occur if the supply of water was unlimited. Figure 1.2-2 shows the region's seasonal potential evapotranspiration.

1.2.5. Problems arising from climate: Duckham and Masefield (1969) state that "apart from some nomadic pastoralism and some very extensive ranching, farming of any type is not at present practicable where (a) the thermal growing season is less

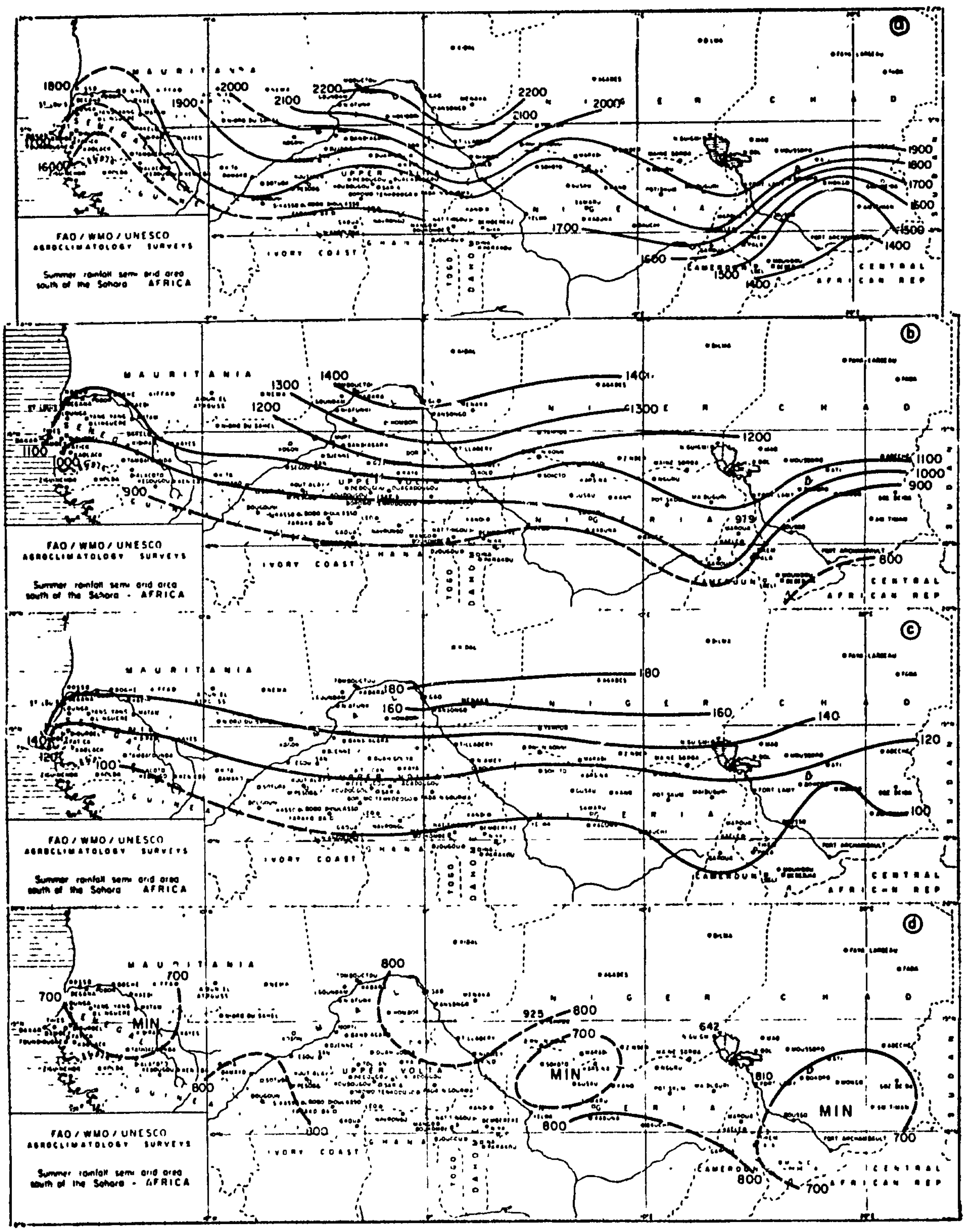


FIGURE 1.2-2 Monthly distribution of potential evapotranspiration (mm): (a) year, (b) July-October, (c) August, (d) November-March.

Source: Cocheme and Franquin 1967.

than 3 months, or where (b) without irrigation, the precipitation or actual evapotranspiration per annum is less than 254 mm, or where (c) the annual potential evapotranspiration is less than 381 mm, or where (d) annual precipitation exceeds 3,050 mm."

Any one of these factors is limiting. Because of the second constraint and the annual rainfall distribution, agriculture without irrigation is impractical north of a line extending approximately from St. Louis, Senegal to the northern shore of Lake Chad. Since Billings (1973) reports, however, that only 102 mm/year of precipitation is sufficient for farming in the area, farming may be possible north of this line. Where Duckham and Masefield are referring to large-scale farm operations, Billings refers to subsistence farming, a distinction which explains this discrepancy in necessary rainfall.

Another climatic problem is that the sudden live squalls occurring north of 11^{o} north cause a great deal of soil erosion. While adjacent land is depleted, organic and mineral nutrients are deposited in pockets or depressions, where they become so excessive that they are inimical to flora growth.

The phenomenon of desertification in the area is due to both climatic and human factors. In some areas, the desert is advancing at the rate of 50 km per year (A.I.D., O.S.T. 1972). Since 1950, 647,200 km of arable land in West Africa have been lost to the desert along a 3,500 km front. A.I.D., O.S.T. (1972) report that desertification can result in less rainfall and higher temperatures, i.e., it can affect regional climates.

One effect of the extremely variable climate is that during wet periods (such as the mid 1960 's) marginal areas are cultivated and herds expand. Then, when a major drought such as the present one occurs, these marginal areas are unfit for cultivation, and overexpanded herds are critically affected. As there is no vegetation to hold the soil, non-marginal cultivated areas also deteriorate, and erosion occurs.

1.3 Surface Water

1.3.1. Rivers: The Niger and the Senegal are the major perennial rivers in the region, in addition to which there are many streams north of 12° that are dry for at least six months of the year.

General characteristics of West African rivers -- South of $12^{\circ}N$ is a series of highland blocks which raise to more than 1,200 m. This feature separates the rivers flowing south from those flowing north. What follows concerns the latter.

The north-flowing rivers originate in areas of high rainfall (two maxima) and steep gradients. Upon entering the region's plains, their gradients are sharply decreased and sediments are deposited, creating, for example, the swamps of the Lake Chad Basin and the inland delta of the Niger River near Segou, Mali. Since sediment deposition causes channel aggradation and division, each river becomes, during the wet season, "in effect, a wide, shallow, slowly moving sheet of water subject to enormous losses by seepage and evaporation." (Ledger 1964)

The retardation effects of vegetation on rainfall runoff explain the fact that runoff pattern does not generally follow rainfall pattern. In southern areas such as Kouroussa, Guinea, where there are twin rainfall peaks, vegetation retards the runoff, causing it to infiltrate during the first part of the rainy season. The soil is then saturated by September, and large runoff occurs, although rainfall from the second part of the rainy season may also infiltrate if the July-August dry season is particularly severe.

Floods on the region's large rivers are due to the superposition of runoff entering the rivers from all parts on the basins at all times of the year. They tend to occur, thus, during years with numerous storms and high annual rainfall. Floods on smaller rivers generally result from a single large storm, which does not necessarily coincide with a year of exceptionally high annual rainfall. Ledger (1964) has found that vegetation cover, along with slope and soil permeability is important in controlling peak flows on small streams and rivers. Vegetation cover is the most important.

Ledger (1964) reports further that West African rivers can be divided into hydrological types based upon hydrograph characteristics (see Figure 1. 3-1). The equatorial type (zone 1) is characterized by two separate periods of high water while the classical tropical type (zone 3) has both high and low water seasons. Between these two are tropical transitional zones (zones 2 and 2a) which have a longer period of high water and shorter period of low water than the Classical tropical type. The sahelian type (zone 4) is similar to zone 3, except with different peaks and low flows. Sub-desert and desert types (zone 5) are characterized by several floods per year, and an average of less than one flood per year respectively. It should be noted again that small streams north of $12^{O}N$ are dry at least six months of the year.

The region's rivers also have short term periods of high and low flows, particularly in northern areas, with short intense squalls and sparse vegetation.

Streamflow Data and Flow Analysis: Shown in Figure 1. 3-2 are the location of river gauges in West Africa. Many have been collecting data since the early 1950 's (Ledger 1964). However, some have been recording longer. Presently, there are about 40 years of data for parts of the Chad and Volta basin, 50 for parts of the Niger basin, and 70 for parts of the Senegal basin. Some basins have been only recently gauged. For example, ORSTROM has less than two year's data on

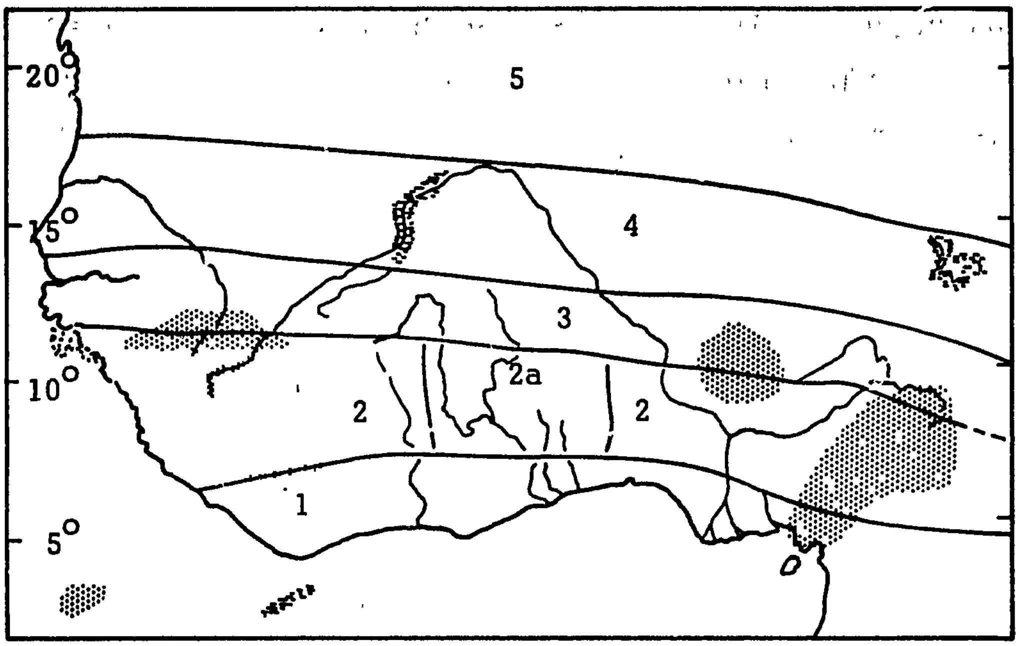


FIGURE 1.3-1 Hydrological regions of west Africa

Source: Ledger 1964. Courtesy of the Institute of British Geographers.

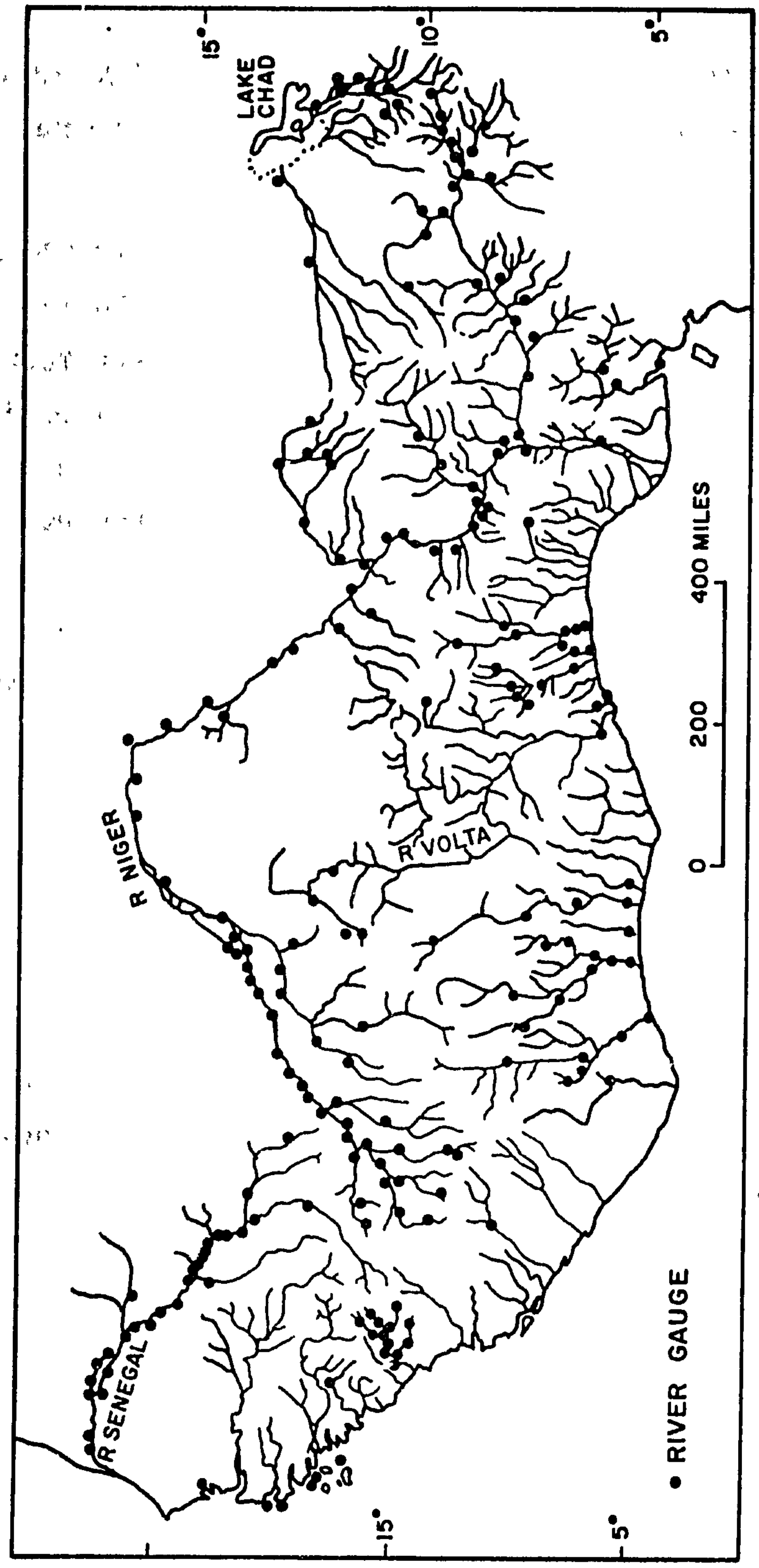


FIGURE 1.3-2 Location of River Gauges in West Africa

Source: Ledger 1964

Permission Applied for Reproduction (Institute of British Geographers) both the Casamance and Gambia Rivers (Chaperon 1974). (For an adequate understanding of a river's regime, at least 25 years of data must exist.)

Experimental Catchments: A related type of data is ORSTROM's experimental catchment data (Dubreuil et al. 1972). The catchments are small areas (perhaps 100 km^2 maximum) throughout West Africa. Each site is studied for several years and all or some of the following information is recorded; rainfall, evaporation, infiltration, soil characteristics, climate, geology, aquifers, vegetation, etc. The location of the sites is in Figure 1.3-3.

Therefore, in some areas, extremely adequate data exists. In addition, Ledger (1964) reports that because of the striking similarities latitudinally between West Africa river basins, it is possible to develop data in "data poor" areas by studying cross-correlation between areas. Therefore, enough information is available for at least broad scale surface water development planning.

Water quality -- Few if any data are available on the water quality of the region's surface water (with the possible exception of Lake Chad). Local pollution problems reportedly exist near major population centers.

1.3.2. Senegal River: The river, which originates in Guinea and flows approximately northwest to St. Louis, Senegal, is 1800 km long. Its basin (shown in Figure 1.3-4) covers an area of about 333,800 square kilometers divided among four countries: Guinea, 30,800 km^2; Mali, 155,000 km^2; Mauritania, 75,600 km^2; and Senegal, 72,400 km^2. Main tributaries are the Bafing, Bakoye, and Faleme rivers which originate on Guinea's Fonta Djalon, where the annual rainfall is 1200 mm or more. Average tributary flows are

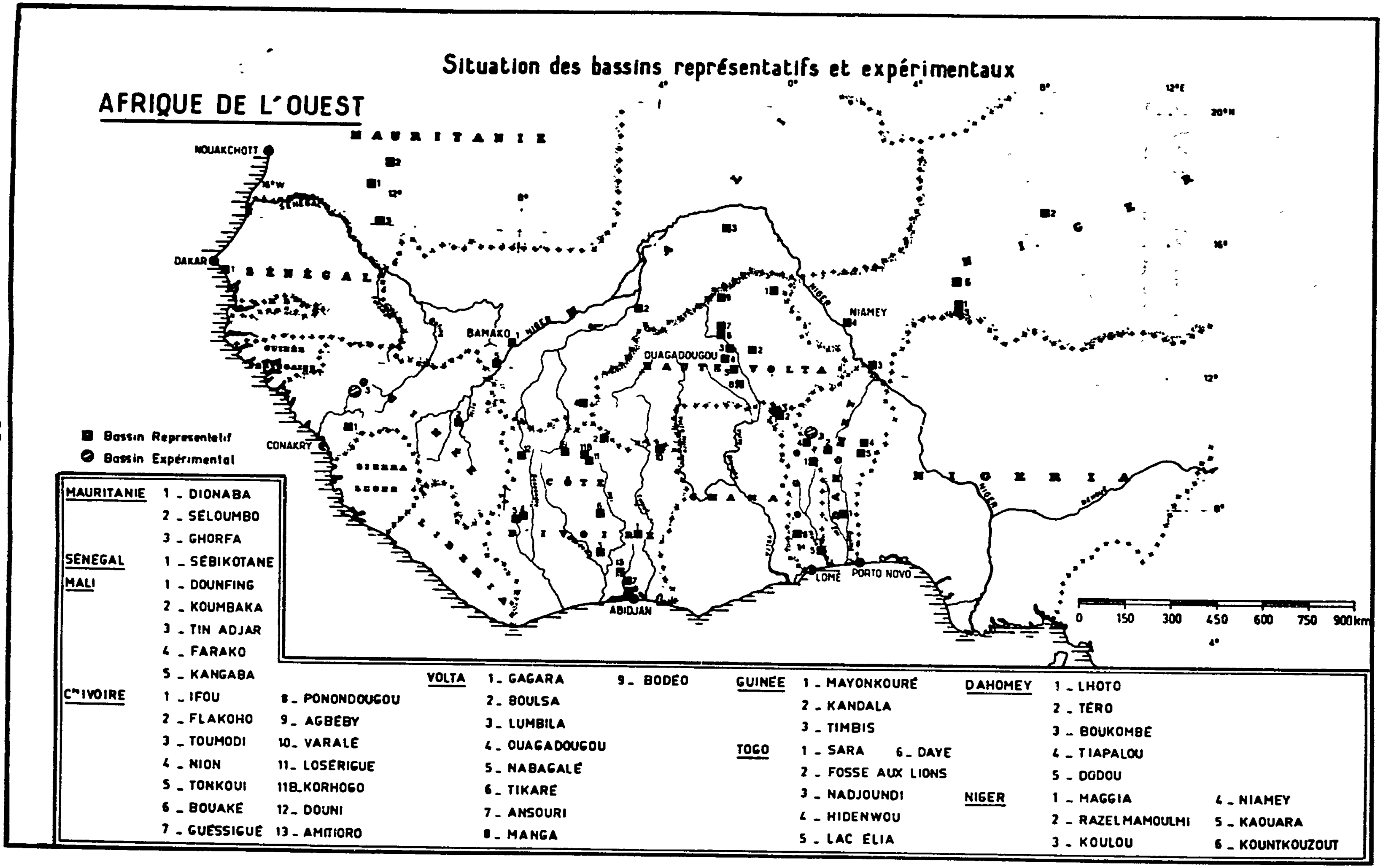


FIGURE 1.3-3 Location of Experimental Catchments of ORSTROM

Source: Dubrueil et al. 1972.

AFRIQUE CENTRALE

Situation des bassins représentatifs et expérimentaux

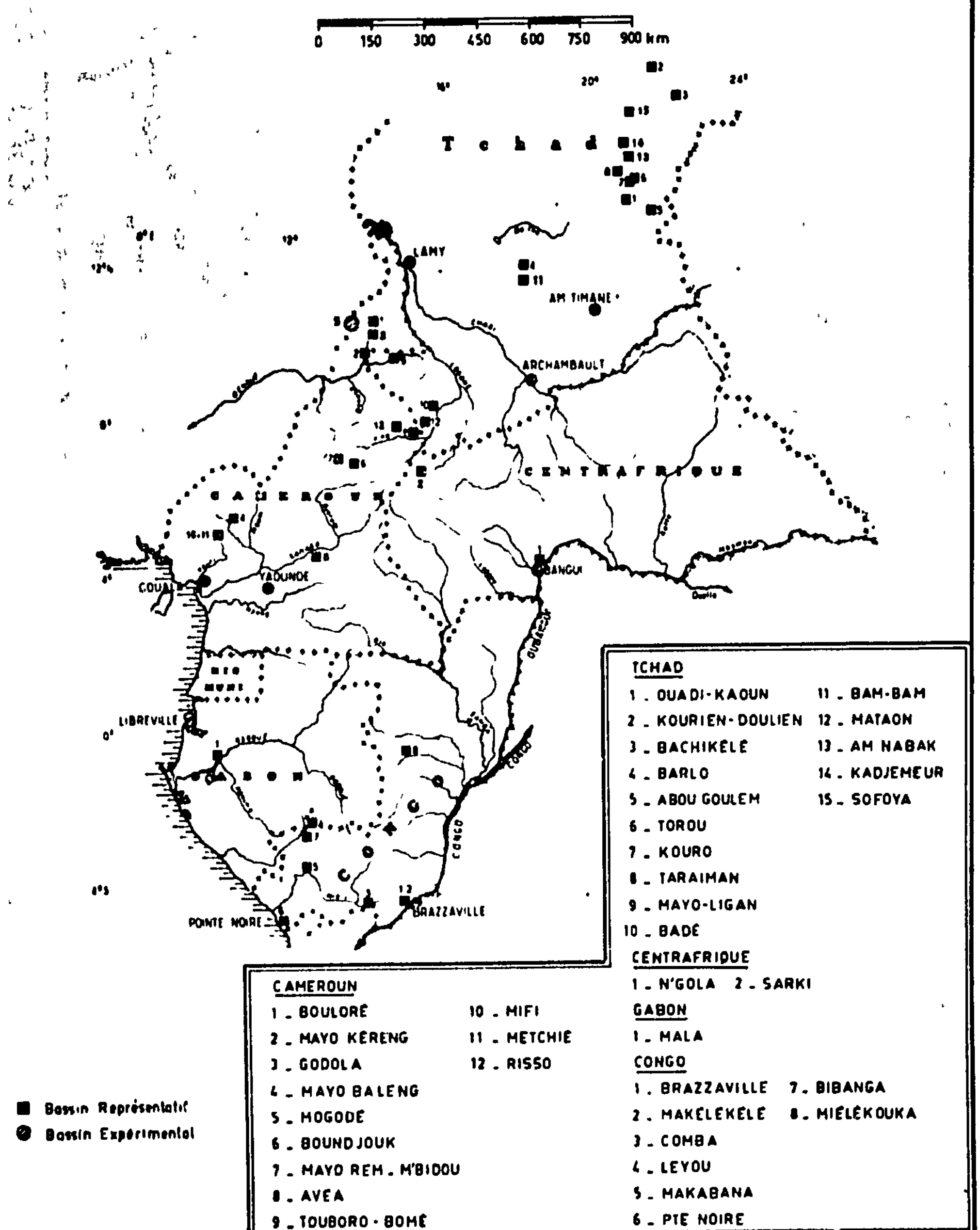


FIGURE 1.3-3 (continued)

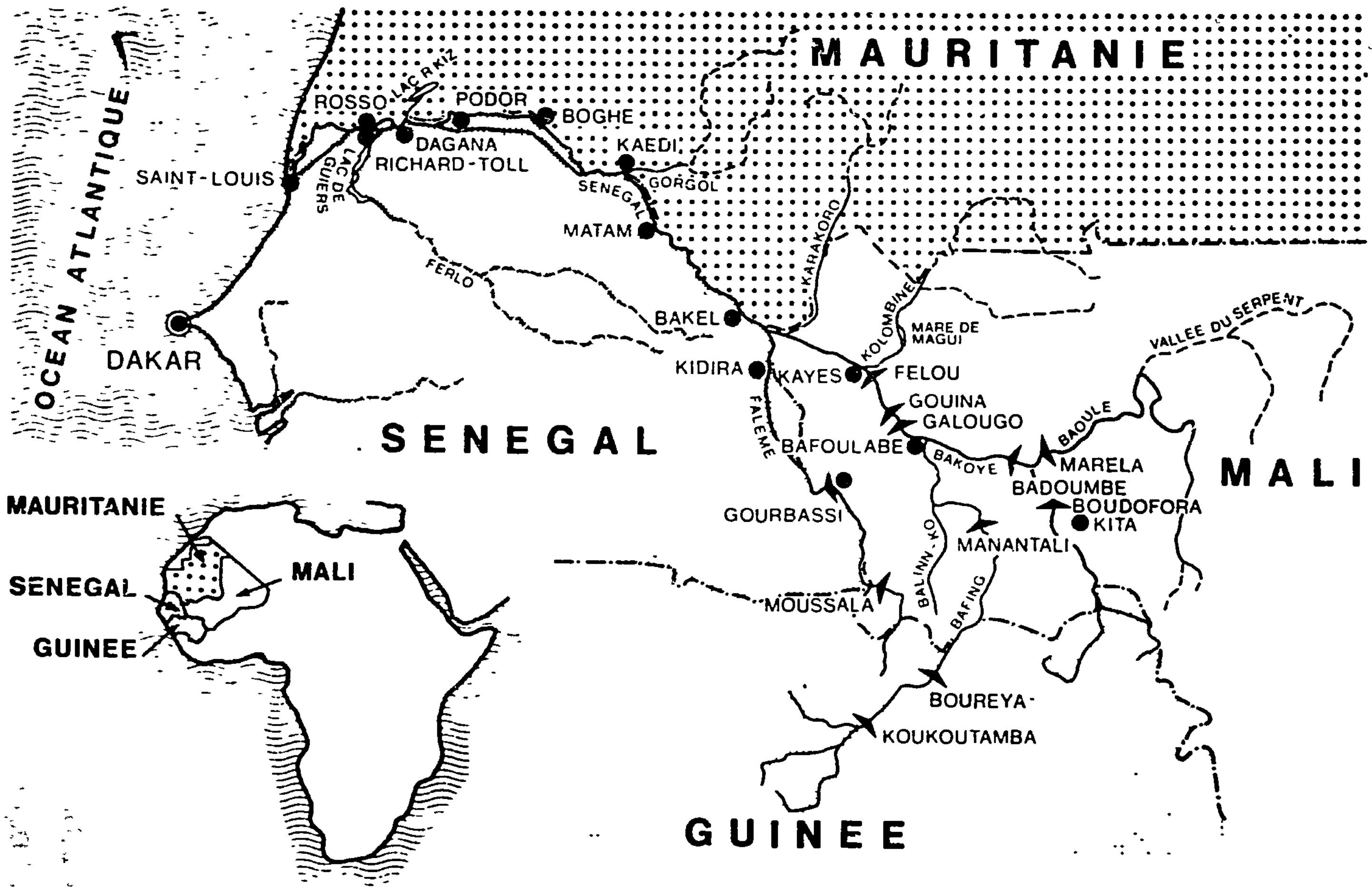


FIGURE 1.3-4 Dam Sites in Senegal Basin

Source: Senegal Consult 1970.

in Table 1.3-1. Average monthly flows at Bakel on the Senegal and at Siramakana on the Baoule are in Table 1.3-2. The peak at Siramakana is one month earlier than at Bakel. No significant inflows occur below Bakel.

Average annual flow at Bakel has varied from 310m^3/sec in 1944-1945 to 1300 m^3/sec in 1924-1925. Flood height at Bakel averages 11 m, decreasing downstream because of storage in the flood plain and in former river branches which are more or less parallel to the main channel. Average flood height is 3.6 m at Dagana, 640 km below Bakel, and less in the Delta area. Lac de R'Kiz and Lac de Guiers also retain part of the flow. The river mouth, diverted by a long coastal sand spit, lies slightly south of St. Louis. Saline water, found under the delta lands, enters particularly far inland during the dry season. Cocheme and Franquin (1968) report saline water is found 100-150 km from the mouth during 5-7 months of the year. IBRD (1973) reports that a flow of 100 m^3/sec is necessary to prevent salt water intrusion.

1.3.3. Niger River: Most of this information is from Dekker (no date). The Niger River basin (6,107,000 km^2) is shared by nine countries: Guinea, 246,000 km^2; Mali, 1,202,000 km^2; Upper Volta, 274,000 km^2; Ivory Coast, 322,000 km^2; Niger, 1,267,000 km^2; Nigeria, 924,000 km^2; Chad, 1,284,000 km^2; Cameroon, 475,000 km^2; and Dahomey, 113,000 km^2.

Furon (1967) describes the river's early history. A long time ago (perhaps one million years), the entire river did not flow to the Atlantic; a lower Niger River entered the Gulf of Guinea, while the upper Niger entered a vast lake in the Tombouctou-Araouane region (Araouane is about 225 km north of Tombouctou). The upper river deposited a vast delta at Macina (about 100 km downstream of Segou, Mali), now the site of the "Office du Niger" irrigation scheme. During flood periods, the river broke its banks and sent

TABLE 1.3 - 1

Average Tributary Flows in Senegal River Basin

Tributary	Flow m^3/sec
Bafing	430
Bakoye*	170
Baoule	60-70
Faleme	200
Karkoro	15-20
Ketiou-Ko	no information
Kolombine	20.

*At confluence with Bafing.

TABLE 1.3 - 2

Average Monthly Flow
(m^3/sec)

MONTH	Bakel	Siramakana
January	142	7
February	83	8
March	46	3
April	20	2
May	10	0
June	110	1
July	584	63
August	2,318	343
September	3,424	265
October	1,689	112
November	571	29
December	256	14

branches far to the north; one of the main branches originated near Sansanding (about 100 km upstream of Macina) and went north to Sokolo, where it split. Eventually, the upper and lower Nigers joined.

Regime of the river -- The Niger River originates in the Guinea highlands (See Figure 1. 3-5). Its monthly hydrograph is single peaked. The flood peaks at Koulikoro in September (see Figure 1. 3-6), at Dire (immediately downstream of the Delta) in December (see Figure 1. 3-7) and at Niamey in January (see Figure 1. 3-8). The hydrograph "flattens out" as the river goes downstream, partly because of natural storage effects in the reaches, but primarily because the Delta greatly retards the flow. Approximately 50 percent of the volume entering the Delta is lost to evaporation and infiltration. The hydrograph at Jebba, shown in Figure 1. 3-9, has two peaks. The predominant white flood in September is due to local tributary inflow, as several major tributaries enter the river downstream of Niamey; the black flood peak in February is due to the flood from upstream.

The major tributaries of the river are seen in Figure 1. 3-10, which shows insignificant inflow between Koulikoro and Niamey. The hydrograph at Douna on the Bani River is shown in Figure 1. 3-11.

1. 3. 4. Lake Chad: Lake Chad is shared by Chad, Cameroon, Niger, and Nigeria (see Figure 1. 3-12). South of the lake, seasonal swamps result from flooding by the Logone/Chari system.

The depth of the lake varies yearly and seasonally. In the north-west portion it is usually from 4-7 m with 10 m maximum in certain areas of the archipelago. In the southeast, alluviation by the Chari River decreases the depth which averages about 3-4 m near the Chari entrance. Along the archipelago the depth is 10 m. Lake Chad Basin Commission (1969) reports that volume is related to depth by

$$\text{vol} = 2.4 \times 10^9 \ (h-278)^2$$

where

vol = volume in m^3

h = depth on IGN reference (4.13 m at Bol corresponds to 282 m IGN).

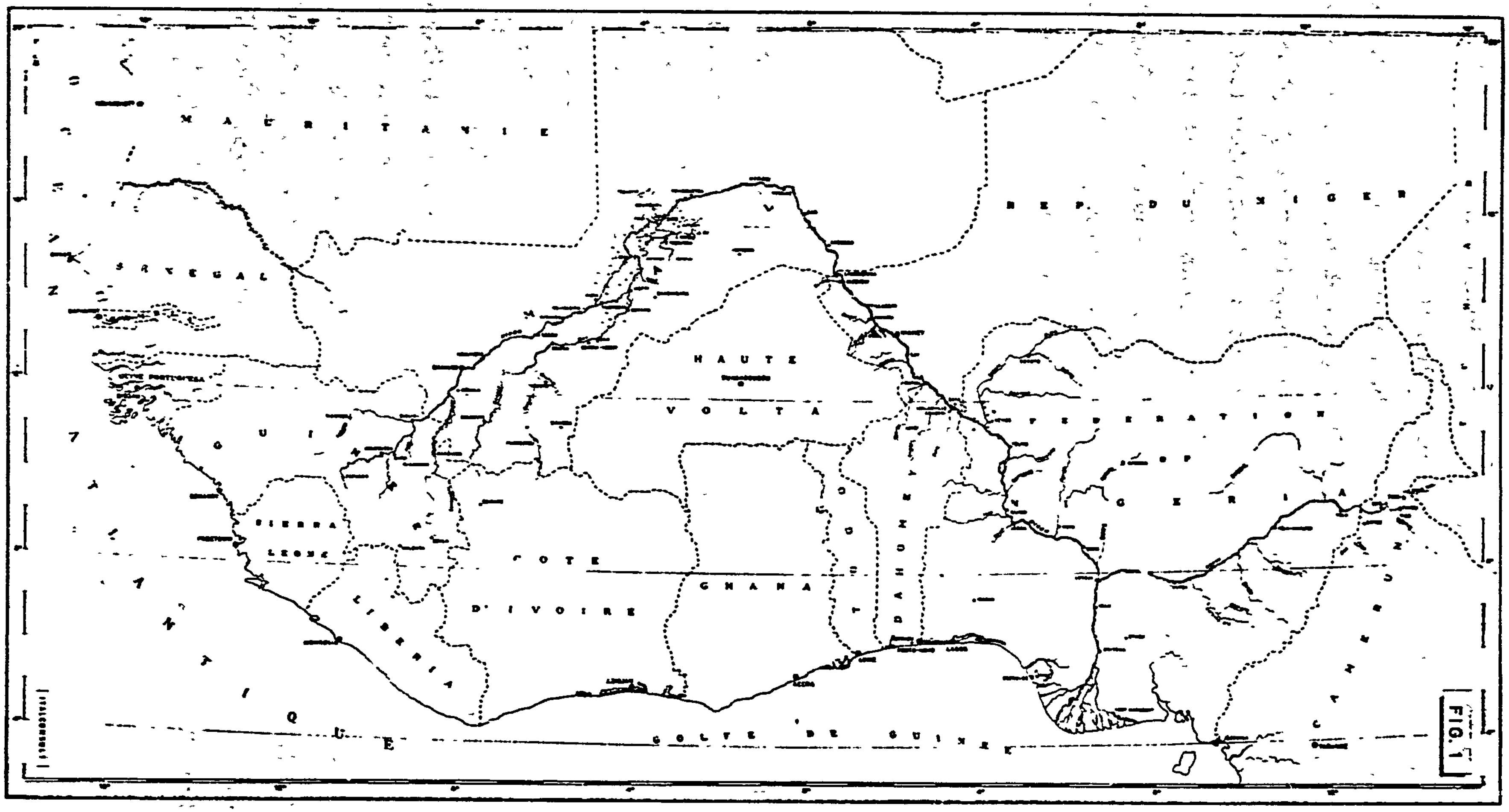


FIGURE 1.3-5 Niger River Basin

Source: Italconsult 1962

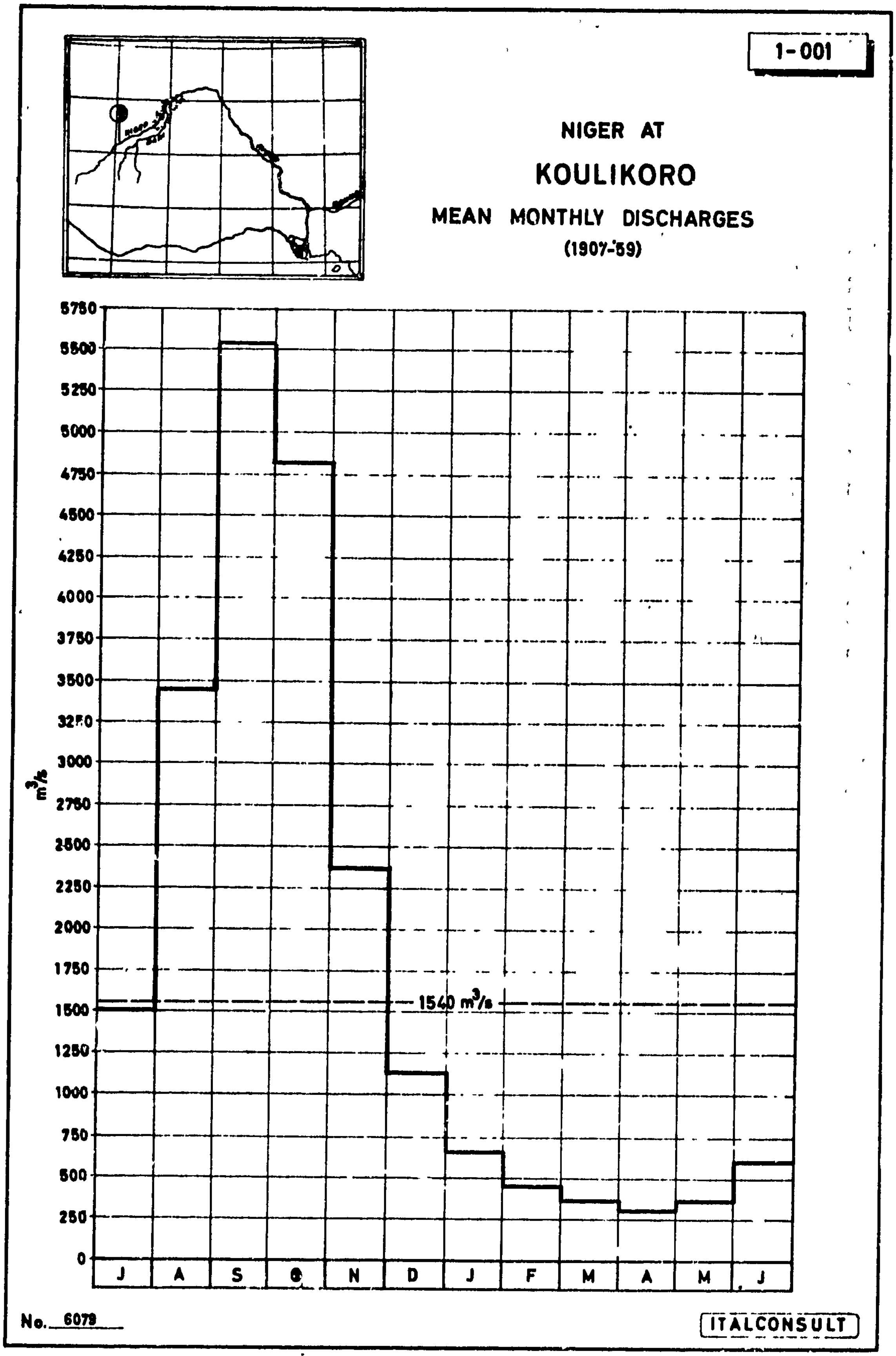


FIGURE 1.3-6 Niger at Koulikoro, Mean Monthly Discharges (1907-50)

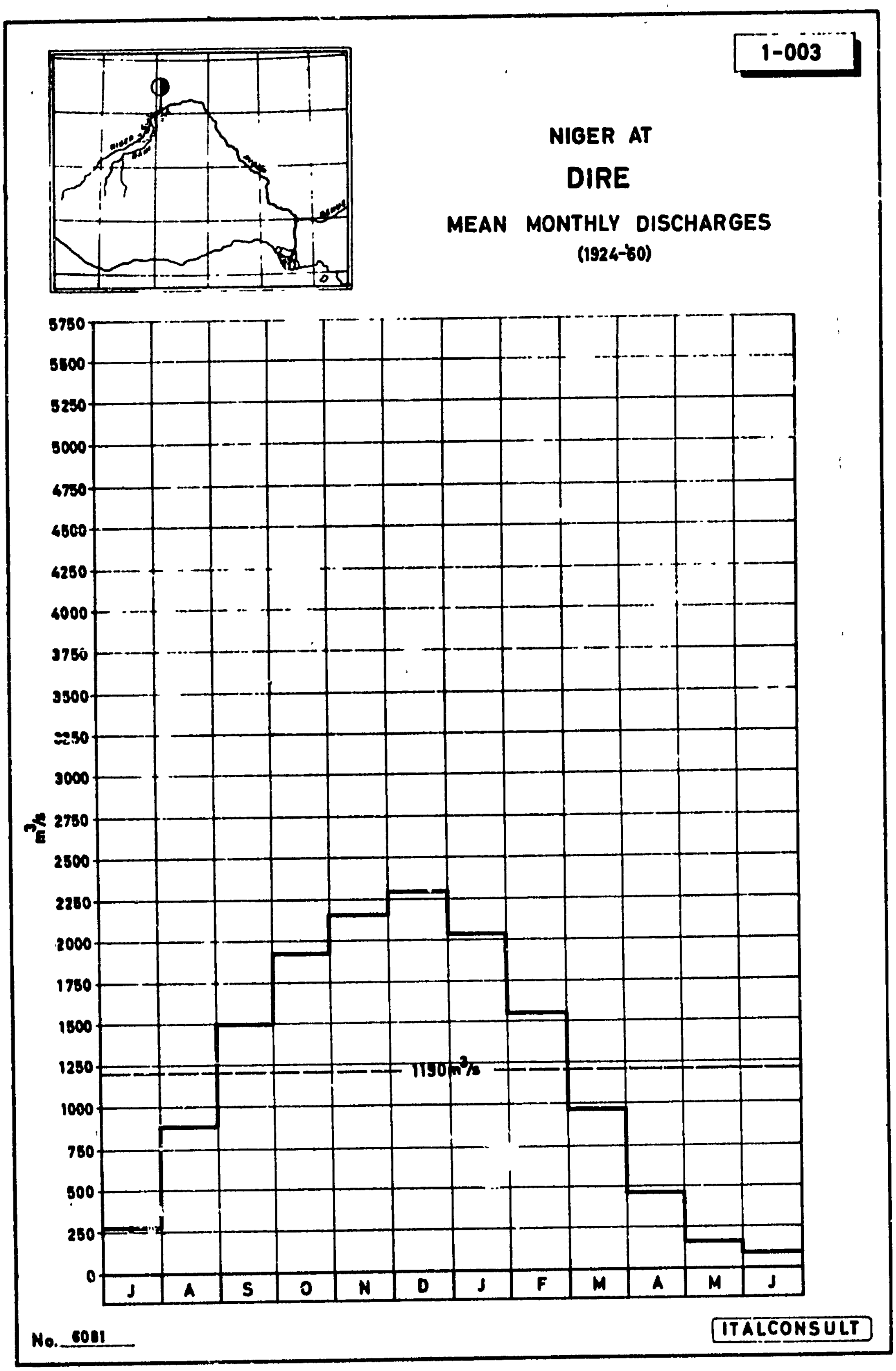


FIGURE 1.3-7 Niger at Dire, Mean Monthly Discharges (1924-60)

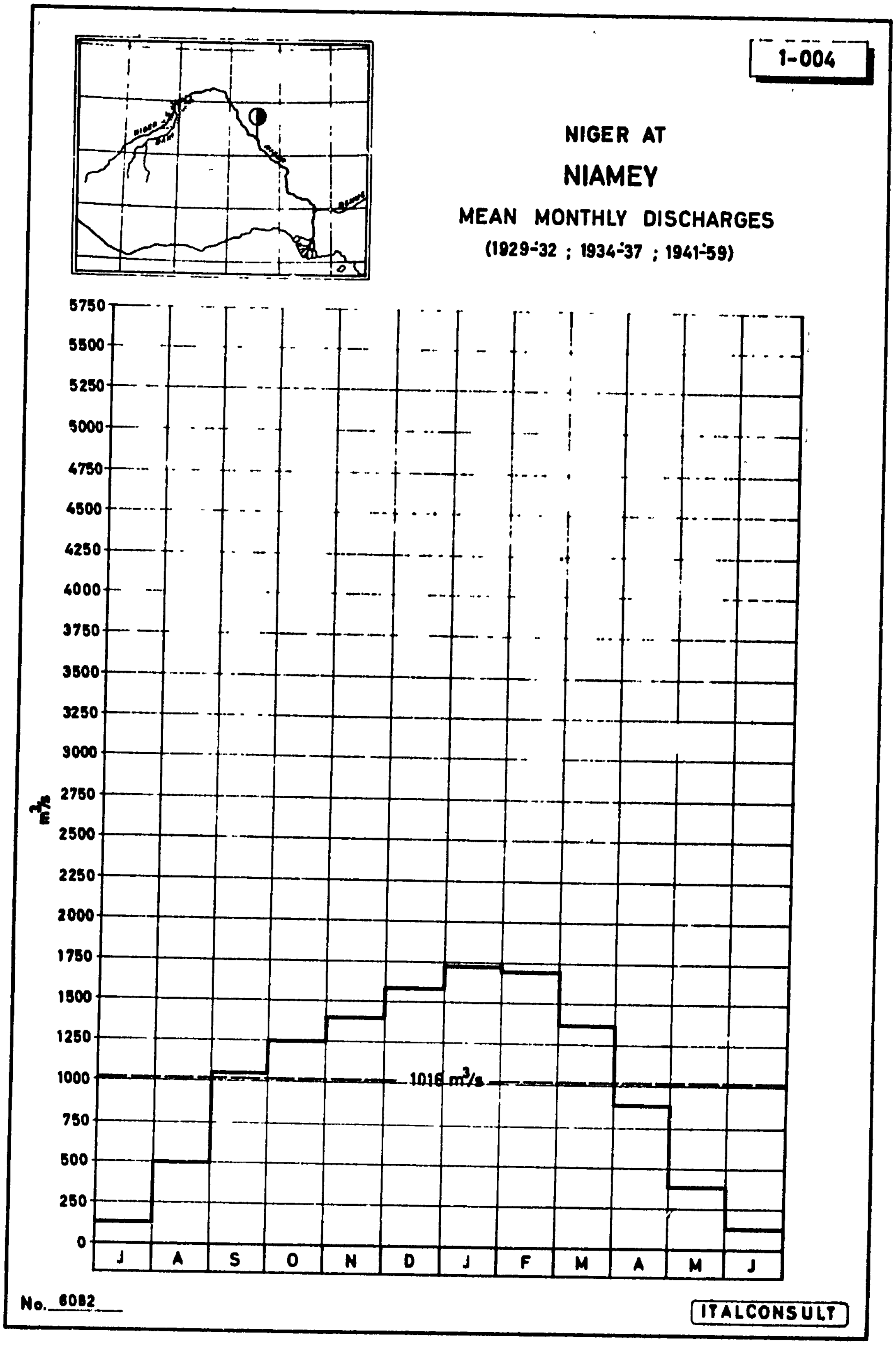


FIGURE 1.3-8 Niger at Niamey, Mean Monthly Discharges (1929-32; 1934-37; 1941-59)

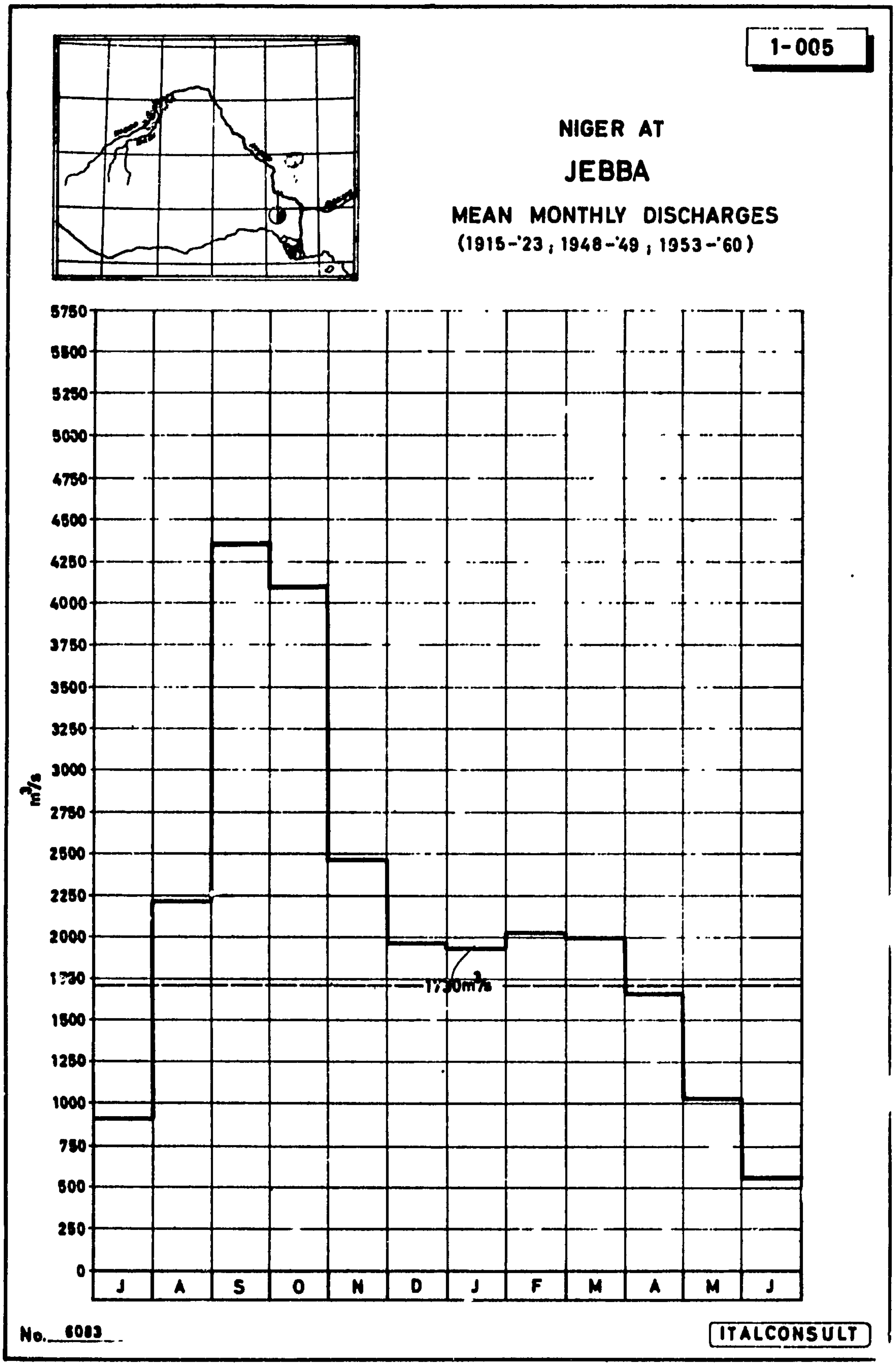


FIGURE 1.3-9 Niger at Jebba, Mean Monthly Discharges (1915-23; 1948-49; 1953-60)

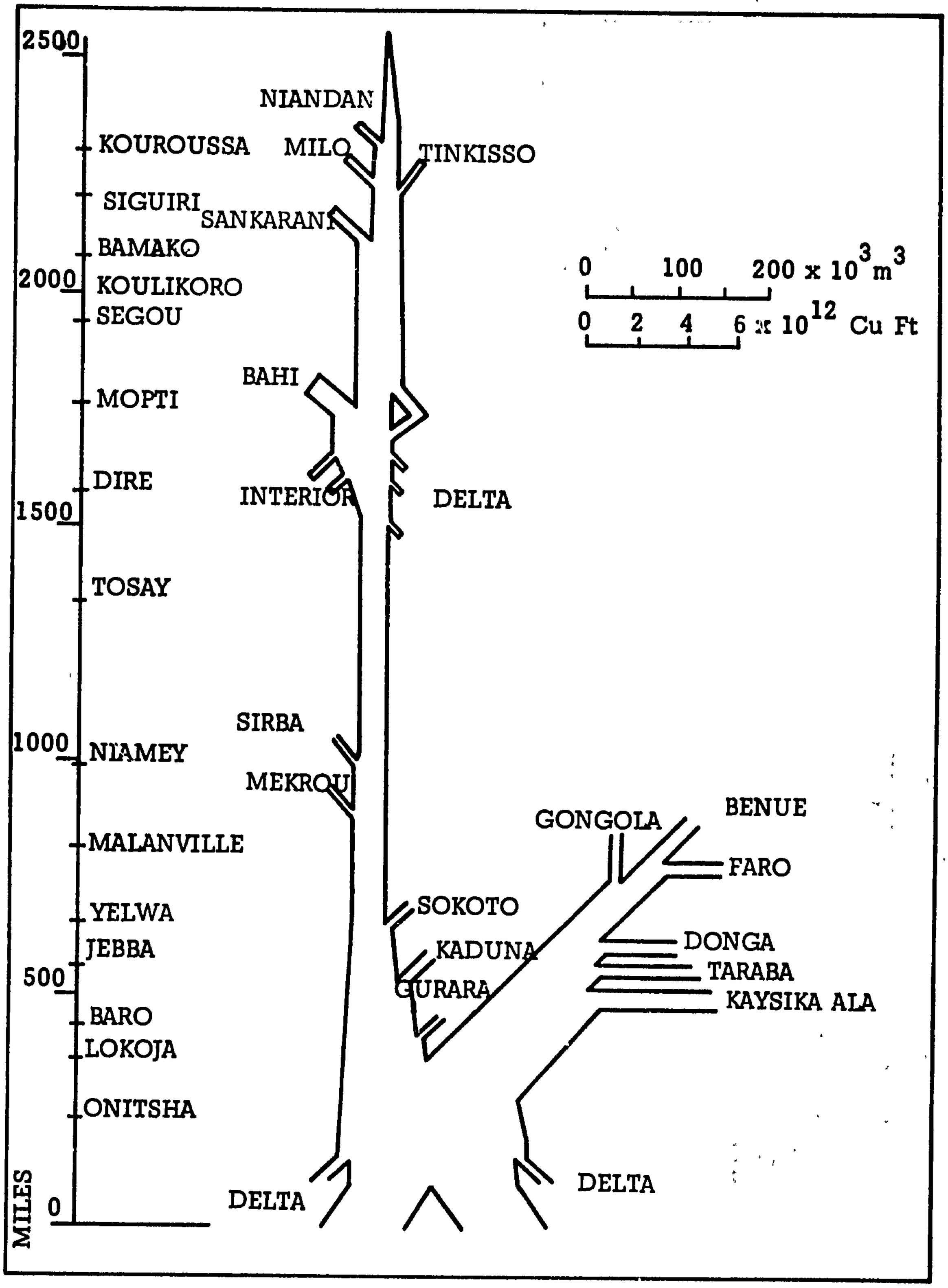


FIGURE 1.3-10 Schematic Representation of the Annual Discharges of the Niger River System

Source: Holmes and Narver 1968

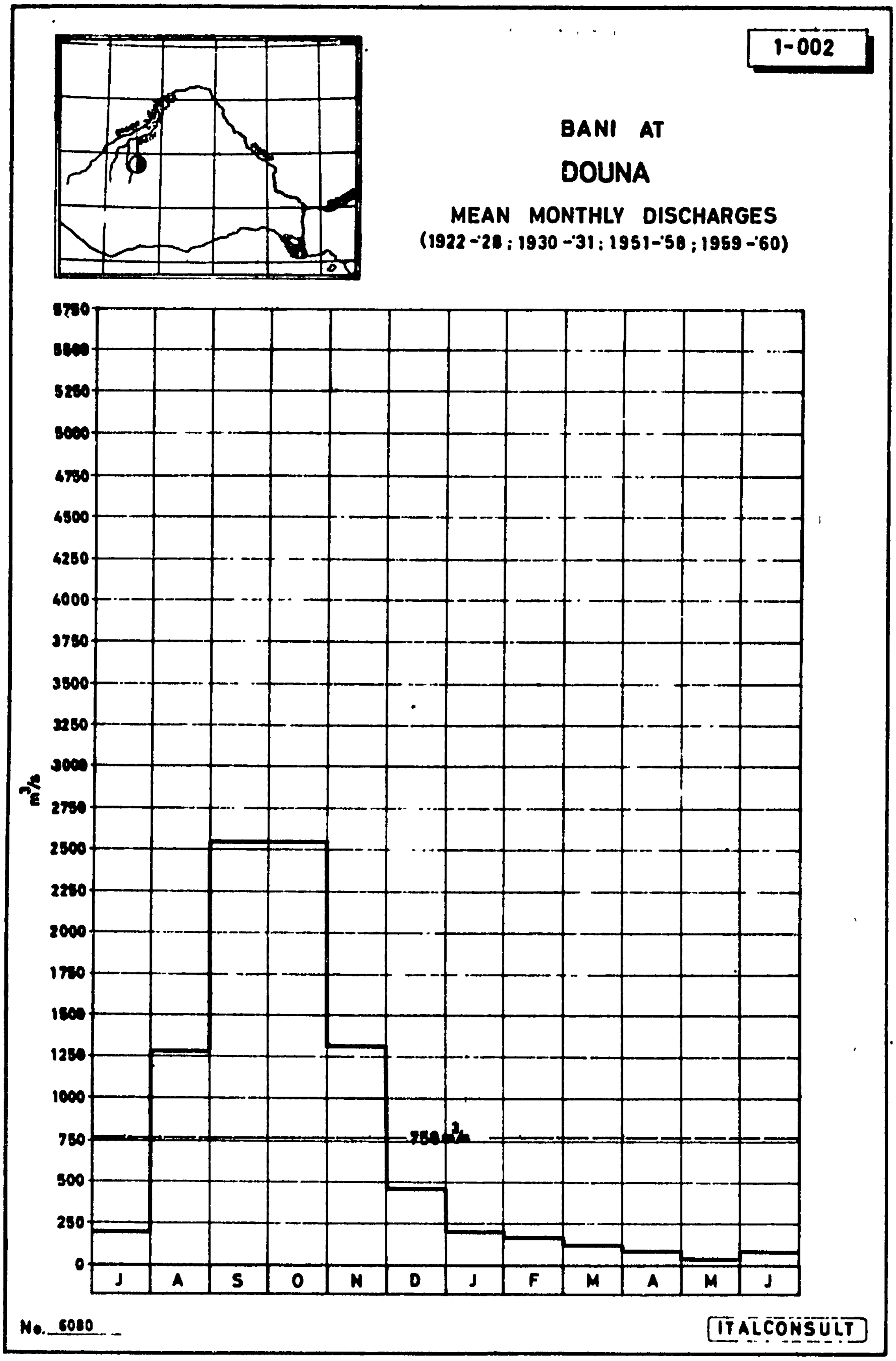


FIGURE 1.3-11 Bani at Douna, Mean Monthly Dischages (1922-28; 1930-31 1951-58; 1959-60)

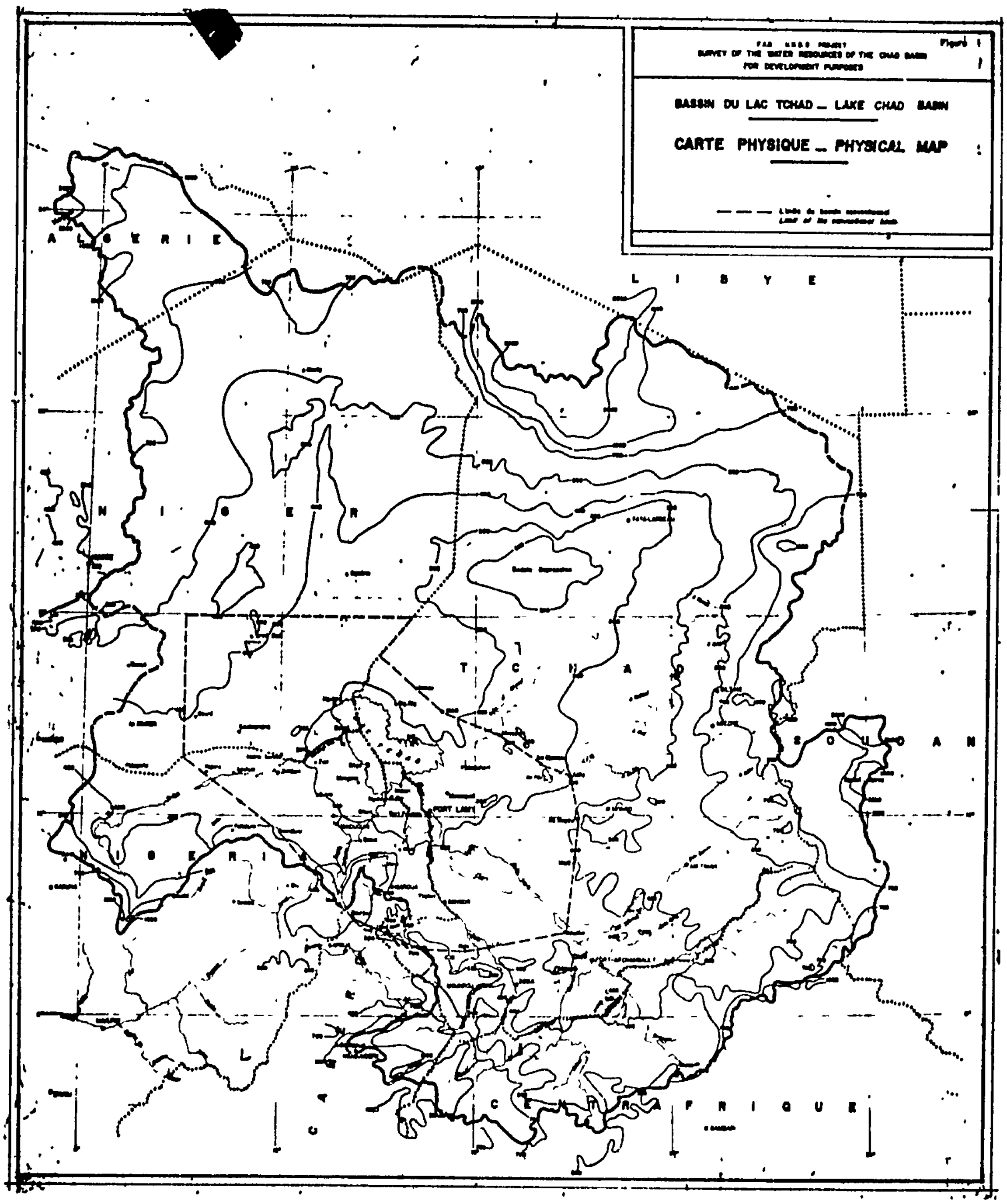


FIGURE 1.3-12 Lake Chad Basin

Source: LCBC 1972.

Concerning seasonal fluctuation, the probability is 50 percent that maximum height will be reached between 25 October and 8 November, minimum between 20 April and 20 May. Over the past 20 years, the annual rise and fall have varied from .13 m to 1.82 m, and from 0.6 to 1.22 m respectively. The rise depends mainly on the magnitude of the Chari flood, the fall on evaporation between January and July.

The lake receives inflows from surface tributaries and rainfall. Average annual inflows from the tributaries are:

Chari	40.4×10^9 m^3
El Beid	2.1×10^9 m^3
Komadugu Yobe	0.5×10^9 m^3
Yedseram	0.1×10^9 m^3

Average annual rainfall on the lake is 351 mm. During the wet period, the average lake surface area is about 11,000 km^2. Because of surface area fluctuations, however, the average influx from rainfall is 3.23×10^9 m^3. Total yearly inflow, therefore, equals 46.3×10^9 m^3.

Losses from the lake occur by evaporation (90 percent) and by infiltration (10 percent) into the banks of the islands in October and November, with some returns to the lake during March and July when lake level is low. Average annual loss by evaporation and infiltration is about 2.3 m. LCBC (1972) reports also a natural overflow for the lake at Bahr-el-Ghazal, which drains northward to the Bodele depressions. No lake depth for beginning of overflow is given, but overflow reportedly occurred when the level was 282.97 m (IGN)* (5.10 m at Bol). LCBC (1972) also notes that the capacity of the overflow channel is small and should not be relied upon to regulate lake levels. If the capacity were increased, however, the channel might serve this function.

* A reference level of 282 m (IGN) corresponds to 4.13 m at Bol.

As described earlier, the lake receives flows from four systems; Chari, El Beid, Yedesam, and Komadugu Yobe. A schematic tributary system is shown in Figure 1. 3-13, representative hydrographs in Figure 1. 3-14. Table 1. 3-3 contains monthly flow data at four stations.

Chari and El Beid system -- The Chari system, composed primarily of the Chari and Logone rivers which join at N'Djamena, is very complicated. Since the major development area is downstream of Miltou on the Chari and Lai on the Logone, these stations can be taken as reference points. Average annual flow near Lai is $17.7 \times 10^9 m^3$; that at Miltou is $30.14 \times 10^9 m^3$. The total inflow from these systems, therefore, is $47.8 \times 10^9 m^3$. However, the total flow at N'Djamena is $40.4 \times 10^9 m^3$ annually. There is obviously a large loss due to natural diversions when flood waters exceed river channel capacities. Part of the annual flood loss from the Logone and Chari overflows from the Logone into the Yaere flood plain, which feeds the El Beid. (The total input to the lake from El Beid is $2.1 \times 10^9 m^3$ annually.) The El Beid thus experiences two peaks, first due to local rains on the Yaere, the second because of overflow from the Logone via the Yaere flood plain. Another important diversion from the Logone is just upstream of Bongor. When the flow between Lai and Bongor reaches 2000 m^3/sec, there is a small overflow to Mayo Kebbi. The Mayo Kebbi runs into the Benue River, a tributary of the Niger River.

Yedseram -- Little quantitative information is available for this system.

Komadugu Yobe -- This tributary, whose drainage area is 147,840 km^2, enters the lake from the west end. Less than 10 percent of its total runoff reaches the lake. The losses are due to

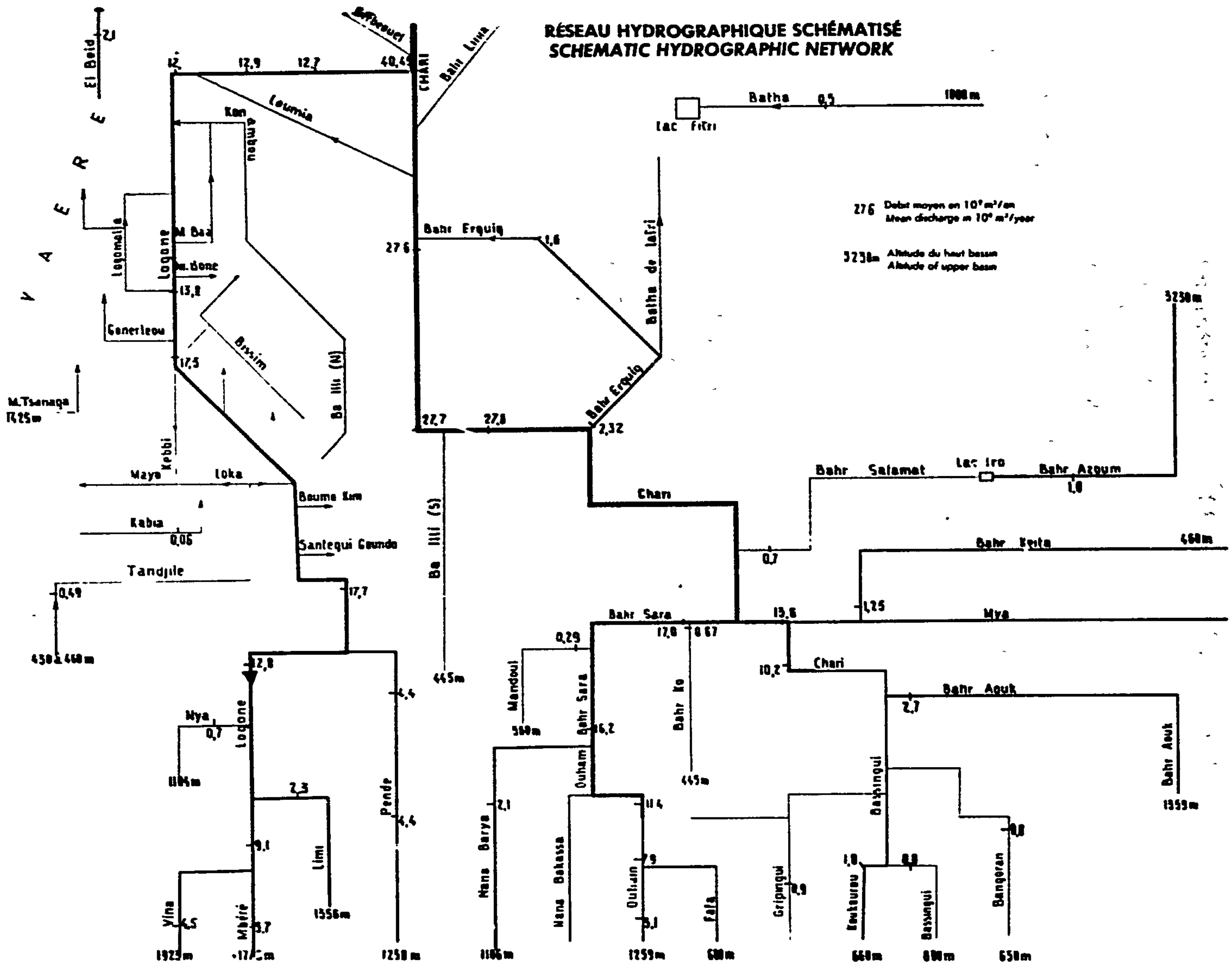


FIGURE 1.3-13 Chad Tributary System

Source: LCBC 1969.

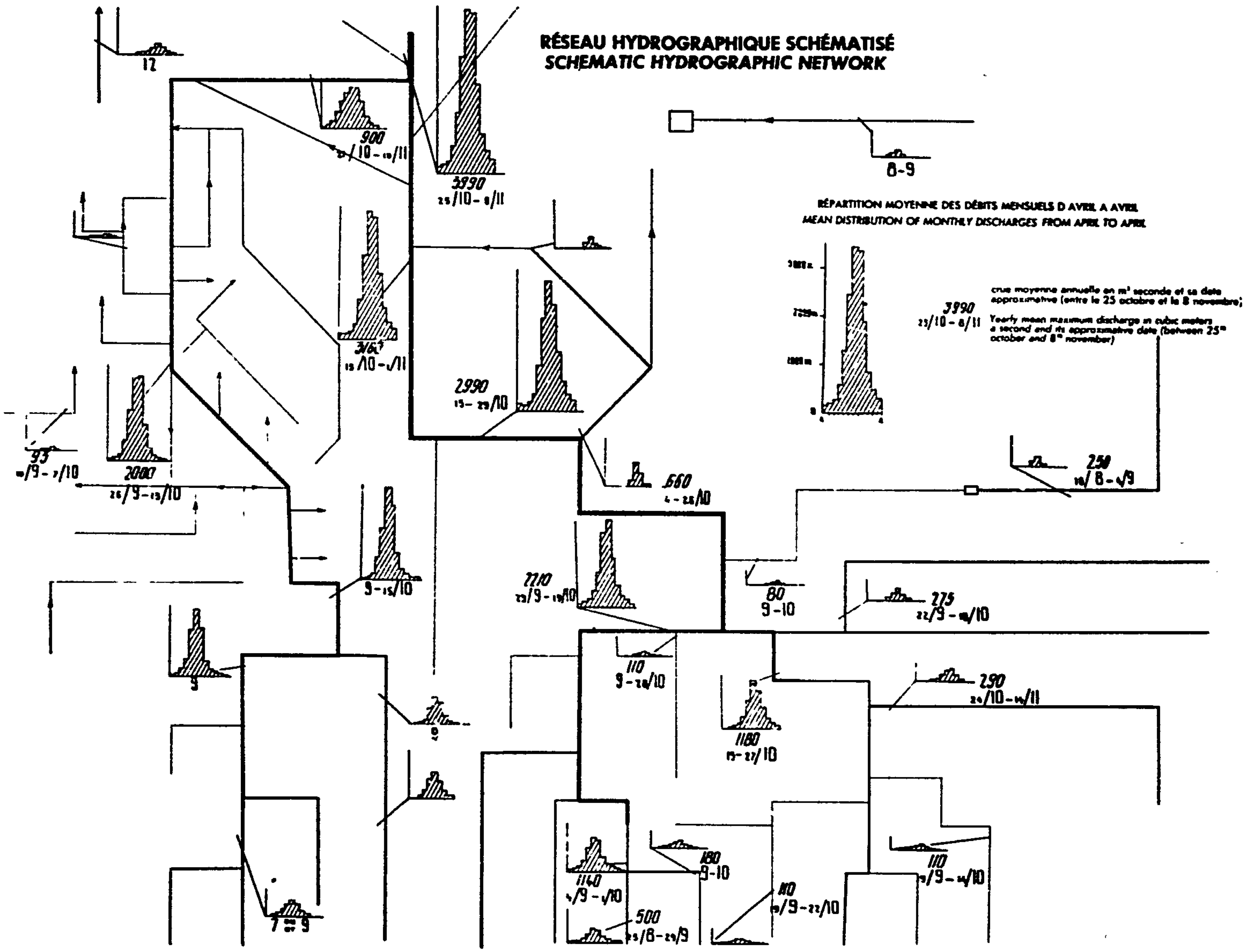


FIGURE 1.3-14 Schematic Hydrographic Network of Chad Basin

Source: LCBC 1969

TABLE 1.3-3

Average Monthly Stream Flows (Approximate) at Selected Stations in the Chad Basin (m^3/sec)

	Lai	N'Djamena	Damasak	Yau
January	130	1,000	30	30
February	60	500	20	10
March	70	450	20	10
April	70	420	20	10
May	90	330	0	10
June	170	420	0	0
July	500	700	20	10
August	1,080	1,440	30	30
September	1,340	2,560	40	30
October	1,560	3,500	50	30
November	520	3,400	60	30
December	210	2,000	65	40

infiltration and overspills. Average monthly discharges at Damasak and Yau are in Table 1. 3-3.

In toto, the rivers entering the lake bring in about 3 million tons of sediment per year and an equal amount of dissolved solids (salt and silica). The water is, however, fresh and suitable for irrigation. Researchers are still searching for an explanation of the lake's freshness.

1. 3. 5. Volta River: The waters of the Volta come from a catchment area above Akosombo Dam of 388, 200 km^2. The more crucial downstream part lies in Ghana (157, 900 km^2). Major tributaries are the Red, White, and Black Voltas (all originating in Upper Volta) and the Oti (originating in Ghana). Because of soil structure and seasonal rainfall the Red and White Voltas are alternately dry or flooded. The Black Volta also fluctuates greatly. These rivers, therefore, are not useful for navigation. The lengths of the three rivers are in Table 1. 3-4. Table 1. 3-5 shows average annual flow at three stations on the Black Volta. Table 1. 3-6 shows average monthly flows at the same stations. The Oti contributes 30-40 percent of the annual flow of the lower Volta.

At Akosombo on the Volta River, Ghana, precise flow records have been kept since 1929. Before the Akosombo Dam was constructed the flow was lowest in March (averaging 28 m^3/sec) and highest in September or early October (averaging 22, 200 m^3/sec).

1. 3. 6. Other Surface Water Resources (USA 1972)

Chad --Other rivers of importance in Chad are the Bahr Sara, chiefly used for floating logs into Chad from the Central African Republic, and the Batha and Bahr-el-Ghazal, which are dry

TABLE 1.3-4

Length of Red, White and Black Voltas

River	Kilometers
Red Volta	270
White Volta	350
Black Volta	450

Source: Measured from Fig. 36 in Grove (19[illegible]7)

TABLE 1. 3-5

Average Annual Flow of Black Volta at 3 Stations
(m^3/sec)

Station	Flow
Kouri	50[1]
Samandeni	14[2]
Ouessa	76[3]

[1]Measurements from graph, Ledger (1964)

[2]UNDP (1973)

[3]UNDP (1973)

TABLE 1.3-6

Average Monthly Flow of Black Volta at 3 Stations
(Approximate Values)
m^3/sec

Month	Samandeni	Ouessa	Kouri
Jan.	5	45	25
Feb.	3	25	10
Mar.	3	15	7
Apr.	3	10	7
May	3	15	7
June	5	25	10
July	8	55	20
Aug.	55	165	55
Sept.	50	310	100
Oct.	20	150	135
Nov.	5	60	130
Dec.	5	30	60

Source: graphs in Ledger (1964) and UNDP (1973) several months of the year. Grove (1967) reports some perennial lakes in the Ounianga area (located southeast of Tibesti Highlands).

Mali -- The Bani River, a tributary of the Niger, is used for transportation in the rainy season.

Senegal -- Other rivers in Senegal are the Casamance and the Saloum.

Gambia River -- This river, which has the largest estuary of Africa, is in Gambia. Over 200 km long, it is navigable for seagoing ships. Grove (1967) reports that even in the dry season, the river is navigable for 500 km and that Kantaur can always be reached by ships drawing 5.2 m of water. Cocheme and Franquin (1968) note that saline water extends 100-150 km inland for five to seven months of the year.

Ceba (or Kayanga) -- Shared by Guinea, Portuguese Guinea and Senegal, this river has an approximate basin surface area of 8,000 km^2.

1.3.7. Water-Related Diseases -- Onchocereiasis disease (transmitted by simulium flies) is a major hindrance to water resource development in parts of the region. The flies inhabit areas with turbulent, well aerated flows. The disease, which causes blindness, is particularly severe in Upper Volta and Niger, but also found in parts of Mali, Senegal, and Chad.

1.4 Surface Water Use and Development

Surface water in the Sahel-Sudan region is used for water supplies, irrigation, electric power generation, transportation, and fishing. Generally, however, the region's surface water resources are not very developed. (Specific projects and uses are discussed later with reference to specific water bodies.) Although water is often the cheapest way to transport bulk goods, lack of good port facilities and a prevalence of river diseases discourage its use.

West Africa has excellent marine and inland fisheries which are an economical source of protein for the area. Because of the upwelling and exposure of rich nutrients, the coasts of Senegal and Mauritania reportedly have some of the richest fishing grounds in the world. These grounds, however, are heavily exploited by foreign nations. Good fishing inland, in the Senegal and Niger rivers and Lake Chad has not yet reached potential because of primitive fishing methods.

All of the development proposals discussed in the following sections are large scale, capital intensive projects. Small scale development using indigenous labor and methods should also be studied for the area. Examples of such projects are the water storage facilities Guggenheim (1974) has successfully installed in Mali. The authors of this report feel small scale development may have several important advantages over large scale development. These include less socially disruptive effects, higher acceptance by the indigenous population, larger marginal yields, better distribution of benefits, greater flexibility, quicker results and less expense. As a form of social experimentation, perhaps small scale development should precede large scale development projects.

1.4.1. Senegal River

Organization pour le Mise en Valeur du Fleuve Senegal (OMVS) -- This organization was formed to replace the Organization des Etats Riveraines du Senegal (OERS), which disbanded because of political difficulties. The Economic Commission for Africa (2) (no date) reports that the OMVS is "charged with the task of promotion and coordination of technical studies and work programs for the joint exploitation of the natural resources of the basin, including such activities as irrigation, river transport, power production and distribution, and mining".

Present situation in valley and delta -- Senegal Consult (1970) reports that there are an estimated 640,000 ha of arable land between Bakel and St. Louis, 510,000 ha in the Valley (Bakel to Dagana), and 130,000 ha in the Delta (Dagana to St. Louis). About 270,000 ha are under cultivation, with only two irrigation projects presently operating in the area. At Richard Toll there are 6,000 - 7,000 ha of rice paddies. However, this land will reportedly be used for sugar cane in the future. In the Delta there are 30,000 ha of rice paddies.

A large amount of land is under flood plain farming. Although areas inundated by floods vary from 230,000 ha (approximately five years out of 100) to 80,000 ha (approximately 95 years out of 100), man-power is available only for cropping a maximum of 180,000 ha. The average amount flooded is 130,000 ha; the remainder of the total area cultivated is under dry land farming. The area also has live-stock activity. The 1950 - 1960 production of the valley and Delta is shown in Table 1.4-1.

Navigation is possible all year round for 2.60 m draft boats from St. Louis to Podor and for 0.65 m boats from St. Louis to Kayes. During the wet season, from July 15 to October 15, navigation for 2.60 m draughts is possible from St. Louis to Kayes, although rocky sills between Ambidedi and Kayes make transportation difficult in this stretch. Navigation is possible in the dry season for 2.6 m boats between St. Louis and Podor because of salt water intrusion in the river to Podor during the dry season. Dekker (no date) reports that the annual fish production in the lower valley is about 15,000 tons, 6,000 tons of which are caught commercially.

TABLE 1.4-1

Agricultural Production in the Valley and the Delta (1950-1960)

Nature of crop	Cultivated surface ha	Yield kg/ha	Annual production 1000 kg
rice	6,000	500 to 3,000	16,800
sorghum	113,000	500 to 600	62,000
millet	96,000	300 to 400	34,000
wheat	12,000	750	9,000
fonio	1,000	250	250
peanuts	15,000	400	6,000
rootcrops	4,000	2,500	10,000

Animals Existing in the Valley and the Delta (1963)

Animals	Number of livestock heads
Cattle	1,230,000
Goats and sheep	1,375,000
Horses	6,000
Donkeys	56,000
Camels	42,000

Future development -- The Basin has been the subject of many studies. Four of the most recent are by Senegal Consult (1970), SOGREAH in 1970, BEYRARD in 1974, and Kulp in 1974. Senegal-Consult studied flow regulation upstream of Bakel. SOGREAH built a mathematical model of the Delta region flows to study the potential for agricultural development there. BEYRARD apparently did a synthesis study for the Valley and Delta region to learn more about agricultural feasibility, particularly economic costs and benefits. Kulp studied appropriate farming systems for the area. Senegal-Consult (1970) presented a staged development scheme based upon regularization of the river's flow for the basin downstream of Bakel. The goals of the development strategy were:

1. to provide water year-round for the Valley and Delta and for navigation from the coast to Mali;

2. to produce hydro-electric power for at least 8,000 hours annually;

3. to provide flood control storage; and

4. to provide for an August flood, so that a gradual transition from flood plain farming to irrigation can take place.

Apparently, Mali, concerned only with power and navigation, was not originally interested in using regulated flows for irrigation.

To provide for regulation and power, Senegal-Consult studied the following sites (see Figure 1.3-4):

Regularization reservoirs	Manantali (Bafing)	
	Gourbassi (Faleme)	
	Galougo (Senegal)	Alternate Sites
	Grand Gouina (Senegal)	
	Badoumbe (Bakoye)	Alternate sites
	Dioubeba (Bakoye)	
Hydroelectric power	Petit Gouina (Senegal)	Run-of-the-river
	Felou (Senegal)	
	Boureya (Bafing)	Storage
	Koukoutamba (Bafing)	

The possible total storage capacities of these sites are shown in Table 1.4-2.

TABLE 1.4-2

Dam Site Characteristics, Senegal River

Principal sites	Mean annual inflow (cu.m/s)	Possible total storage (1000 million cu.m)
Badoumbe	167	10
Boureya	273	10
Dioubeba	170	10
Galougo	616	35
Gourbassi	165	5
Grand Gouina	616	35
Koukoutamba	202	5
Manantali	377	35
Secondary sites		
Bindougou	290	2
Boudofora	75	2
Moussale	97	3
Marela	80	3

Source: Senegal-Consult (1970)

To determine water requirements for irrigation, Senegal-Consult (1970) assumed double cropping of rice on 60 percent of the irrigated area and high value crops such as cotton, peanuts, sugar cane, fruits, vegetables, and fodder crops on the remaining 40 percent. The rice yields were assumed to be 3,000 Kg/ha crop; yields of other crops were average values obtained from similar irrigation studies. The monthly water requirements of all irrigated areas are shown in Table 1.4-3.

The staged development scheme proposed by Senegal-Consult (1970) is:

1. Construction of the Manantali Dam with a useful volume of $10.0 \times 10^9 m^3$ and total capacity of $11.1 \times 10^9 m^3$. For nine years out of ten, the dam would provide a regulated flow of 300 m^3/sec at Bakel and an output of 100 mw during 8,000 hours. In the first stage of development, it would permit a flow of 100 m^3/sec at Bakel and an artificial August flood of 3,500 m^3/sec. These figures would then change to 200 m^3/sec at Bakel (and 3,000 m^3/sec in August), and finally to 300 m^3/sec at Bakel. The final stage would allow irrigation of about 230,000 ha in the region and provide year-round navigation for 1.4 m draft boats between St. Louis and Ambidedi (this draft corresponds to self-propelling cargo-boats with a load capacity of 350 tons).*

2. Construction of the Gourbassi Dam with a useful volume of $1.5 \times 10^9 m^3$ and total volume of $2.1 \times 10^9 m^3$. This dam would increase regulated flows at Bakel to 400 m^3/sec and increase power output by 13 mw.

3. Construction of the Galougo Dam with a useful volume of 30×10^9 m^3 and total volume of 31.9×10^9 m^3. The dam would increase regulated flow at Bakel to 500 m^3/sec and total output to 300 mw.

*Vol 1B, pp 5-6 reports that 300 m^3/sec is sufficient for 1.4 m draft. Vol 4, pp 4-34 reports that 400 m^3/sec is necessary for this draft.

TABLE 1.4-3

Monthly Water Requirements for Irrigated Areas
(Percentage of Total Annual Requirement)

Month	Percent
January	6.5
February	7.0
March	9.5
April	10.5
May	11.0
June	10.5
July	9.5
August	8.0
September	7.0
October	8.0
November	6.5
December	6.0

4. Construction of hydroelectric power plants to meet possibly increased power demands. One of the four sites previously mentioned or a new site in Guinea would be chosen.

Senegal-Consult reports that a flow of 550 m^3/sec at Bakel would provide irrigation for 480,000 ha in the valley and delta, and a draught of at least 1.4 m between St. Louis and Ambidedi.

It is interesting to note Senegal-Consult's (1970) determination by simulation that the maximum regulated flow which would be achieved 90 percent of the time at Bakel is 700 m^3/sec (average annual unregulated flow is 771 m^3/sec). This regulation would require dams at Galougo, Manantali, and Gourbassi with useful volumes of 30 x 10^9 m^3, 30 x 10^9 m^3, and 7 x 10^9 m^3 respectively.

Senegal-Consult (1970) describes two operating policies of the dam. The first requires operation of reservoirs to maintain a constant year-round flow at Bakel, except during August when an artificial flood would be released. This procedure is intended for the early stages of development, when flood plain agriculture is still being practiced. The policy would apply only to the Manantali Dam. Regulation would be as follows:

Constant Discharge Sept. - July (m^3/sec)	Constant Discharge August (m^3/sec)
100	3,500
200	3,000

The second operating policy is intended for later development. The reservoir, filled during the rainy season, would be used to maintain a minimum discharge of 300 m^3/sec year-round at Bakel. Surplus would be stored.

Senegal-Consult (1970) describes the consequences of river regulation and water management as follows:

"The future reservoirs in the Upper Basin, with a storage capacity equal to several annual flow volumes, and the resulting flow regime modifications obviously will affect a number of factors of vital importance for the basin. Primary consequences are discussed briefly below. Secondary effects of the regulation, such as industrial and social development processes also to be credited to the regulation, are not mentioned.

Climate:

Owing to the regulation of the river, the valley will no longer be flooded except in extraordinary situations. Therefore, evaporation in the valley will decrease greatly. On the other hand, the large water surfaces in the Upper Basin will cause an increase of evaporation in this region. There is no doubt that these changes will influence the micro-climate in both regions, but it is difficult to evaluate the respective consequences.

Evaporation:

There is still an unknown aspect with regard to surface evaporation from the future reservoirs. It is quite possible that the soils of the reservoir basins, especially of the large, shallow lake of Galougo, will absorb water until saturated, and retain it when the reservoir water level falls. This soil water would then be returned to the atmosphere by evaporation. Hence, the total water losses would increase considerably.

Wildlife:

River regulation will bring about important changes in wildlife conditions. The natural living conditions of fish, game, birds, and reptiles will be subject to far reaching modifications. Fishery conditions will be particularly affected in the Delta where certain fish species may disappear. On the other hand, fishing will probably become a major activity on the great reservoir lakes. Water fowl are supposed to abandon the Delta, finding better conditions on the Upper Basin lakes.

Vegetation:

The great forests in the valley, now protected, which owe their existence to the annual inundation by the Senegal, will disappear. These forest areas will be converted into steppes.

River Morphology:

The behavior of the rivers will be influenced by the modifications of the flow regime downstream of the reservoirs, especially on the Senegal below Bakel. A number of consequences unfavorable to navigation will arise, such as increase of sand banks, decrease of channel curvature radii and local increases of flow velocities.

Groundwater:

In the valley the water table will be higher during the dry seasons and lower during the rainy seasons, as compared to the present situation.

Agriculture:

A substantial quantity of suspended material is deposited on the land by the annual floods. This material represents a natural fertilizer. The quantity of suspended material will greatly decrease as a result of river regulation.

This beneficial effect of natural flooding will disappear despite inundations caused by artificial floods.

Projects:

Agricultural development projects, in operation or under consideration (Richard Toll, Lake R'Kiz, Gorgol Valley) will become problematic due to constantly high water levels in the valley."

Analysis of Senegal Consult (1970) Statements -- This section analyzes two statements made by the Senegal-Consult (1970) report.

Statement 1: 300 m^3/sec regulated flow at Bakel would provide irrigation for approximately 230,000 ha and year-round navigation between St. Louis and Ambidedi for 1.4 m draft boats.

Assume that one half of the irrigated area is between Bakel and Boghe (reach 1), with the other half between Boghe and St. Louis (reach 2). (The latter is a reasonable assumption since approximately 1/5 of the area's total arable land is between Dagana and St. Louis, which is approximately 1/5 of the distance between St. Louis and Bakel. Boghe is approximately half way between Bakel and St. Louis.) Therefore, half of the water is withdrawn in the first reach, the other half in the second. Width varies; in the valley (Bakel to Dagana) from 200 to 400 m, and in the delta from 400 to 500 m. Width from Bakel to Boghe thus varies from 200 m to about 330(based upon distance ratios); from Boghe to St. Louis it is from 330 to 500 m.

Assuming that the average annual evaporation rate between Boghe and St. Louis is 2,470 mm - (that at Richard Toll) - that the rate between Boghe and Bakel is 2,330 mm (that of Matam) and that the lengths of reaches 1 and 2 are approximately 270 km, the annual evaporation losses from the water surface are:

Reach 1: $270 \times 10^3 m \times 2.33\ m \times \frac{200 + 300\ m}{2} = 167 \times 10^6 m^3$

Reach 2: $270 \times 10^3 m \times 2.47\ m \times \frac{500 + 330\ m}{2} = 276 \times 10^6 m^3$

or

Reach 1: $5.3\ m^3/sec$

Reach 2: $8.8\ m^3/sec$

Manning's Equation (Linsley and Franzini 1972) can be used to find average depth, since steady uniform flow is assumed in each reach.

$$Q = VA = \frac{A(1.49)}{n} R^{2/3} S^{1/2}$$

n = roughness

R = hydraulic radius = $\frac{w.d}{w+2d}$

S = slope

A = cross-sectional area

An n value of 0.030 can be used for Reach 2 and a value of 0.04 for Reach 1. 0.030 corresponds to "clean and straight natural channel", and 0.04 to "winding, with rocky beds".

Senegal-Consult (1970) reports that the river surface slope is $\frac{3 \times 10^{-2} m}{10^3}$ between Bakel and Matam and $\frac{1 \times 10^{-2} m}{10^3}$ in the Podor-Richard Toll section. These calculations will be used as the slopes in Reaches 1 and 2 respectively. Manning's equation can now be used to find the minimum discharge needed for a navigation depth of 1.4 m. The minimum net discharge necessary in Reach 1 is calculated to be

64 m^3/sec, that in Reach 2, 74.9 m^3/sec. However, since a minimum of 84 m^3/sec (8.8 + 74.9) is needed when entering Reach 2, at least this amount is necessary at the end of Reach 1. To account for evaporation loss, the minimum amount entering Reach 1 has to be 90 m^3/sec (84 + 5.3). However, since 100 m^3/sec is needed for salinity control in the delta, and the total evaporation loss from Bakel to St. Louis is approximately 15 m^3/sec, 115 m^3/sec will be used as the minimum discharge at Bakel needed for both navigation and salinity control. Therefore, a flow of 300 m^3/sec would 185,000 ha of irrigation at the rate of 1 liter/sec/ha and navigation.

Statement 2: A regulated flow of 550 m^3/sec at Bakel will provide irrigation for 480,000 ha and year-round navigation from St. Louis to Ambidedi for 1.4 m draft boats.

Since water requirement would be

480 m^3/sec	+	115 m^3/sec	=	505 m^3/sec
irrigation		navigation		

statement 2 is apparently in error.

To check on the navigation possibility between Bakel and Ambidedi, we can assume that the Manatali dam (the only one assumed built) provides at least 200 m^3/sec of the 300 m^3/sec regulated flow at Bakel. (The report never states that if 300 m^3/sec is guaranteed at Bakel, whether this requires inflows from other tributaries.) Therefore, the flow at Kayes (downstream of Manantali Dam) is a minimum of 200 m^3/sec year-round, which corresponds to a depth of about 2m (on the basis of the rating curb at Kayes.) (Senegal-Consult 1970) Since Ambidedi is just downstream of Kayes, we can assume the same approximate depth at Ambidedi and downstream to Bakel. Therefore, navigation will be possible between St. Louis and Ambidedi.

Cost of regulation of 300 m^3/sec -- The cost of obtaining a regulated flow of 300 m^3/sec is the sum of the construction and operational costs of the Manatali Dam. The breakdown of costs is shown in Table 1.4-4. Power output would be 100 mw (150 mw installed).

Maximum amount of land that can be irrigated -- Senegal-Consult (1970) reports that 700 m^3/sec is the maximum regulated flow that can be obtained at Bakel. To be determined is how much land can be irrigated with enough water left over for year-round navigation between St. Louis and Ambidedi for 1.4 m draft boats. Since a flow of 700 m^3/sec, using 115 m^3/sec for navigation and salinity control leaves 585 m^3/sec for irrigation, the potential is 585,000 ha.

A regulated flow of 700 m^3/sec is obtained by the construction and operation of the Galougo, Manatali, and Gourbassi dams. The power output would be 190 mw. The cost of the dams alone would be \$340 million (1969). This figure includes construction and project administration. Operation and maintenance charges for the dams are approximately \$619,000 annually, not including financial charges. We can assume that the hydroelectric costs will be approximately double those at Manatali when the regulated flow is 300 m^3/sec. Capital costs would thus be \$40 million while engineering costs and interest during construction would be \$9.0 million. Operation and maintenance, etc. would cost \$1,462,000 annually. (For summary, see Table 1.4-5)

Delta dam -- There is a proposal to construct a dam in the delta 30 km upstream from St. Louis, which would permit double cropping of up to 60,000 ha by stopping salt water intrusion and storing water. If such a dam were built after the Manatali dam, its only function would be to act as a salt water barrier (IBRD 1973). IBRD (1973) reports that the two projects would permit the irrigation of

TABLE 1.4-4

Estimate of the Investment Required for Manantali Project

Cost Summary

Construction Cost of Dam and Power Plant	Alternative described: Complete Development (10^6 US $)	Regulation Alone (10^6 US $)	Power plant Alone (10^6 US $)
Dam and ancillary structures			
Civil engineering	59.5	59.5	—
Equipment	3.7	3.7	—
Power plant and switchyard			
Civil engineering	4.5	—	4.5
Equipment	15.6		15.6
Total	83.3	63.2	20.1
Construction Cost of Roads, Railways and Villages			
Site access roads	4.6	4.6	—
Rerouting of roads	5.8	5.8	—
Reconstruction of villages	0.9	0.9	—
Total	11.3	11.3	—
Development Cost	7.0	5.5	1.5
Interest During Construction	13.8	10.8	3.0
Cost of Project Administration		national	
Total Investment	115.4	90.8	24.6

Source: Senegal Consult (1970)

(Con't.)

TABLE 1.4-4(cont.)

Manantali Project: Estimate of Annual Charges

	Alternatives described		
	Complete Development (US $)	Regulation Alone (US $)	Power plant Alone (US $)
Financial Charges	6,355,000	4,635,000	1,720,000
Replacement Charges	152,000	49,000	203,000
Operating Costs	315,000	45,000	270,000
Maintenance Costs	438,000	180,000	258,000
Cost of Administration		national	
Total	7,260,000	4,809,000	2,451,000

Source: Senegal Consult (1970)

TABLE 1.4-5

Cost Summary for Senegal Basin Development

Regulated amount	Power output	(1969) Cost of dams (millions$)	(1969) Annual OMR costs of dams($)	Cost of power plants $ million	Annual OMR costs of power plants	Maximum irrigation hect.*,**
$300m^3$/sec	100mw	90.8	674,000	$24.6	731,000	185,000
$700m^3$/sec	190mw	340	619,000	$49.0	1,460,000	585,000

* With year round navigation from Ambidedi to St. Louis (1.4m) (maybe 1.4-2.0m)

** 1 liter/sec/ha consumptive rate

330,000 ha. This amount seems doubtful because this would require at least 300 m^3/sec of flow at Bakel leaving no water available for transportation and other needs. However, IBRD (1973) does report that about 100 m^3/sec are needed at St. Louis to prevent salinity problems. Since a regulated flow of about 75 m^3/sec in Reach 2 will be maintained for navigation, an additional 25 m^3/sec might as well be provided for salinity control. Therefore, the delta dam would not increase the maximum amount of hectares that can be irrigated in the valley and delta.

Senegal valley potential with no regulation -- As discussed ed previously, little irrigated cropping now exists in the region. The minimum flow needed for navigation from St. Louis to Ambidedi is about 90 m^3/sec at Bakel. However, as assumed earlier, 115 m^3/sec is required to keep salt water out. Navigation for 1.4 draft boats is therefore feasible from June through January under average conditions, while the amount of irrigated agriculture possible from July through December would be about 140,000 ha. Since a large amount of water would flow by when the flood peaks at 3400 m^3/sec protective levees would be needed.

Senegal Valley potential under drought condition -- Senegal-Consult (1970) reports that it is possible to provide a ma[illegible]um regulation of 700 m^3/sec -- or 90 percent of the annual average (770 m^3/sec) at Bakel year-round. As Kirshen (1974) discussed in the Niger River Basin Update, average 1970—1972 drought conditions resulted in 44 percent less flow in the Niger River. Under drought conditions, the average flow that could be provided at Bakel would therefore be ([0.56 x 770] x 0.90) 390 m^3/sec, which would provide enough flow for year-round navigation, and 275,000 ha of year-round irrigation. Since the Manatali and Gourbassi dams together would, under normal conditions, provide a regulated flow of 400 m^3/sec at Bakel, and if 275,000 ha will provide enough produce in normal years, it may be best to build only these two dams.

Although the delta dam's potential under drought conditions is unknown, it could conceivably be 60,000 ha (its maximum amount in good years) since drought flows could easily provide the small water requirement for this amount.

Potential with drought and no regulation--During the flooding of the Senegal River in July-August, 1972, water did not rise above the lower, minor river bed. At the time crop planting covered only a tenth of the area. If we assume this is one-tenth of the arable land flooded, only 64,000 hectares were flooded. It could also mean one-tenth of the maximum area, or 23,000 ha, flooded; assume it is 64,000 hectares.

A minimum flow of 115 m^3/sec is needed at Bakel for navigation and salinity control; the rest could be used for irrigation. Using the 1913* hydrograph at Bakel as representative of the 1970-1972 average, Table 1.4-6 shows that 135,000 ha could be irrigated from July through November. We previously determined that the flow at Bakel should be about 90 m^3/sec for navigation from St. Louis to Ambidedi. If the 1913 records at Bakel approximate the conditions of the 1970-1972 average, navigation for 1.4 m draft boats under these conditions would be possible seven months of the year, from June through December (see Summary, Table 1.4-7).

Summary -- There are several valuable observations:

1. The combination of the Manantali and Goubassi dams, providing a regulated flow of 400 m^3/sec at Bakel, would function equally effectively under drought and normal

*1913 was also a drought year.

TABLE 1.4-6

1913 Flows at Bakel

Month	Flow (m^3/sec)
January	64
February	30
March	10
April	4
May	10
June	120
July	330
August	704
September	918
October	680
November	251
December	121

Source: Senegal Consult (1970)

TABLE 1.4-7

Potential in Senegal Basin under Drought Conditions

	Agriculture	Transportation
No regulation	64,000 ha (Flood Plain) or 135,000 (July through November with pumping)	1.4m draft navigation from St. Louis to Ambidedi, June through December.
Maximum regulation (700 m^3/sec at Bakel)	275,000 year-round	1.4m draft from St. Louis to Ambidedi, year-round
Delta dam only	60,000 ha (?) double cropping (see text)	?

conditions.

2. The Delta Dam will serve no real purpose as long as 100 m^3/sec is provided in Reach 2. Since 75 m^3/sec is needed for navigation, 100 m^3/sec can probably be provided with no loss in benefits.

3. Under conditions of irrigation development without regulation, the valley and delta can irrigate roughly the same amount during normal years and drought years (see Summary in Table 1.4-8).

1.4.2. Niger River

Niger River Commission -- The Commission was established in 1964 to promote studies and programs concerning exploitation of the basin's resources. All nine basin states are commission members.

Transportation -- Transportation, an important activity on the river, serves as both a coastal-inland and an inland-horizontal link. Stretches of the river vary in navigability. According to Dekker's (no date) detailed discussion, the reach from Kouroussa, Guinea to Bamako, Mali is navigable from July to December; Bamako-Koulikoro is not navigable because of rapids; and, navigability from Koulikoro to Ansongo varies seasonally as below:

Koulikoro-Fanchon	June - January
Fanchon-Segou-Markala	All year, because of barrage at Markala
Markala-Mopti	July - March
Mopti-Ansongo	January - April

No mention is made of draft. Navigability follows the seasonal flood downstream.

From Ansongo to Labezanga the river is apparently navigable,

TABLE 1.4-8

Summary of Navigation and Irrigation in Senegal Valley and Delta

Controls	Average Year[X] Agriculture	Average Year[X] Transportation	Drought[X] Agriculture	Drought[X] Transportation
Manantali Dam ($300\ m^3/sec$, Bakel)	185,000 ha (YR)	(YR)	185,000 ha (YR)	(YR)
Galougo[+], Manantali, Gourbassi dams ($700\ m^3/sec$, Bakel)	585,000 ha (YR)	(YR)	275,000	(YR)
No regulation	130,000 ha (FP) or 140,000 ha** (July - December)	June - January	64,000 (FP) or 135,000** (July - November)	June - December
Delta Dam only	60,000 ha (?) double cropping (see text)	?	60,000 ha (?) double cropping (see text)	?

Symbols: YR = Year round
FP = Flood plain

[X] for 1.4 m draft boats from St. Louis to Ambidedi

[+] Maximum regulation

** Irrigation

while from Labezanga to Tillabery, Niger, navigation is impossible because of rapids. From Tillabery to Yelwa navigation is possible from about September to May. The river is navigable year-round from the Kainji Reservoir to the sea. In fact, low profile river steamers can reach Niamey from the sea at least eight months a year. Shown in Figure 1.4-1 is a graph depicting navigable periods on the Niger River between Kouroussa, Guinea, and the ocean. On the basis of known average monthly discharge at stations in Koulikoro, Dire, Niamey, Yelwa (considered the same as at Jebba because of insignificant inflow between them), and Jebba (taken from Italconsult, 1962), we can determine minimum flows necessary for navigation at these stations (see Table 1.4-9). It should be noted that the data in Figure 1.4-1 and Table 1.4-9 were developed before construction of the Kainji Dam, which permits year-round navigation from Yelwa to the ocean (see later discussion of Kainji Dam). "Rise" and "Fall" in Table 1.4-9 corresponds to the rise and fall of the river.

Agriculture -- The present activity is described below. Most of the data is from Italconsult(1962).

Guinea - Estimated to be little agricultural activity of any sort.

Mali -

Genie Rural	Controlled	90,700 ha above Delta
	Flooding	103,000 ha below Delta

Present irrigation schedule is:

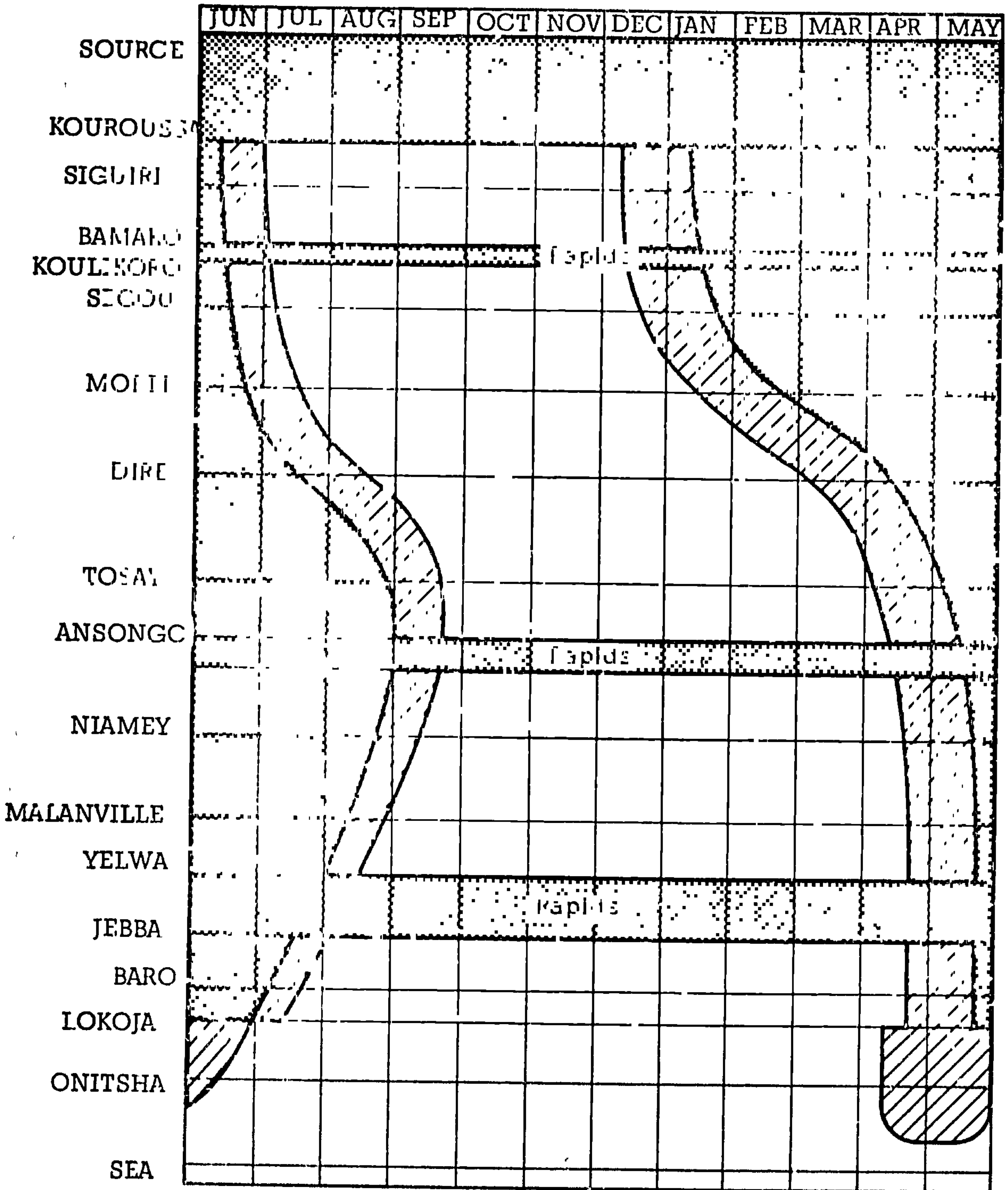
Aug. $3 \times 10^3 m^3/ha$

Sept. $4 \times 10^3 m^3/ha$

Oct. $3 \times 10^3 m^3/ha$

Estimated present usage is arrived at by adding 1962 usage and proposed development during the following five-year plan.

There are no data on Delta use. The major crops are rice and floating rice.



Open to Navigation for ships with a draft ≥ 1.3 m

Restricted navigation for ships with a draft of 1-1.3 m

No navigation. Available depth less than 1 m

FIGURE 1.4-1 Navigability of the Niger

Source: Holmes and Narver 1968.

TABLE 1.4-9

Navigation Discharges, Niger River

Location	Minimum discharges (m^3/sec)			
	1 m - 1.3 m Draft		1.3 m Draft	
	Rise	Fall	Rise	Fall
Koulikoro*	625	675	1,000	1,125
Dire (rapids)	250	475	875	1,250
Niamey	500	450	1,025	875
Yelwa	1,550	1,025	2,225	1,700
Jebba	875	1,025	1,550	1,700
Baro to Lokoja+	900	900	1,000	1,100
Lokoja to Ocean+	1,200	1,200	1,500	1,500

* Carlo Lotti and Co. (1972)

\+ Italconsult (1962)

Office du Niger

The 1962 plus proposed five-year plan amount total 98, 800 ha. The 1962 irrigation schedule (35, 700 ha) is:

January	-	50 m^3/sec
February	-	50
March	-	40
April	-	40
May	-	40
June	-	90
July	-	75
August	-	110
September	-	110
October	-	75
November	-	75
December	-	50

Niger

In 1962, only about 1, 000 ha were irrigated. Dekker (no date) also reports that there is little water usage.

Dahomey

No data; little if any present use.

Nigeria

Italconsult (1962) reports that 12, 400 ha are now being partially irrigated.

The projected activity is described below.

Guinea

There are no data, except that there is potential on the Tinkisso, Milo, and Niandan Rivers.

Mali

Genie Rural - Italconsult (1962) reports that Genie Rural has long term plans to irrigate a total of 117,700 ha above and 132,300 ha below the delta. No information on potential is available.

Office du Niger - Italconsult (1962) reports that Office du Niger plans to irrigate 254,800 ha by the year 2000. The only information on potential is that the French originally proposed to irrigate 2-5 million acres.

Niger

Dekker (no date) reports that of 30,000 ha of potentially irrigatable basins, only 14,600 ha can be economically developed. He also notes that 12,000 ha of the terraces are below 7 m and that 16,000 ha are above 7 m. Niger plans to develop 2,600 ha of the terraces and 3,000 ha of the basin.

Dahomey

There are no data.

Nigeria

Dekker (no date) reports that below the Kainji Dam, potential for 80,000 ha for irrigation and 160,000 ha for dry land agriculture now exists, while in the Sokto Valley, there is potential for 190,000 ha (for Summary, see Table 1.4-10).

Fishing -- Since the area for fisheries is 11,000 km^2 at flood, the Niger River is an important fishing area. While it has produced 45,000 tons annually, its potential has hardly been realized.

Floods -- Flooding occurs in both Nigeria and the upper part of the basin. Mali suffers periodic flood damage from sudden rises in tributaries from Guinea. Establishment of a flood warning system and improved land use in the Highlands might reduce the damage.

The arrival of the "black flood" in Nigeria can now be predicted fairly accurately on the basis of past hydrologic observations. Since operating policy of the Kainji Dam is connected to flood arrival,

TABLE 1.4-10

Summary of Present, Future and Potential Irrigation in Niger Basin

Country (and Location)	Present	Future	Potential
Guinea	?	?	Tinkisso, Milo, Niandan
Mali			
G.R. above Delta	90,000ha	117,700ha	?
below Delta	103,000ha	132,000ha	?
Office du Niger	98,800	254,800ha	2-5 million?
Niger			
Basins	1,000ha (terraces and basins)	3,000	30,000 (perhaps only 14,600 econ. feasible)
Terraces		2,600	28,000 (perhaps only 12,000 econ. feasible)
Dahomey	?	?	?
Nigeria	12,400ha		
Between Jebba and Lokoja			80,000 + 160,000 Dry land
Sokoto Valley			190,000 ha

any major modification of the flood could affect the dam's economic results. The reservoir has considerably reduced downstream flooding and made more farm land available.

Existing dams and power plants in Niger River Basin -- Sites of existing dams and power plants in the Niger River Basin are shown in Figure 1.4-2.

1. Sotuba Power Plan, Niger River, Mali

This plant has no effect on the river's regime. Dekker reports it has a capacity of 10.4 mw (36 x 10^6 kwh). If Selingue Dam is c nstructed, installed capacity can be increased to 21 mw (120 x 10^6 kwh). A conflicting report from Italconsult (1962), written before the plant was built, says that output is 1.7 mw at low flow (36 m^3/sec, head = 7.65m) and 4.7 mw at high flow (6200 m^3/sec, head = 3.65 m).

2. Markala Power Plant, Niger River, Mali

Dekker (no date) reports that this plant has 1 mw (4 x 10^6 kwh) installed capacity, which could be increased to 5 mw (32 x 10^6 kwh) if Selingue is constructed.

Italconsult (1962) reported (before the plant was built) two 650 KV units installed and operated 8 months of the year. With maximum discharge of 12.5 m^3/sec, the plant has a head of 5.5 m.

This plant has no effect on the regime of the river.

3. Kainji Reservoir and Dam, Niger River, Nigeria

This project, in Nigeria just north of Jebba, was recently completed. The dam, rock-filled with a concrete center section, is the first project of a series to be constructed for electricity production in this area. It has volume capacity of 15 x 10^9 m^3, useable capacity of 11.5 x 10^9 m^3, and average surface area of 1.250 x $10^3 km^2$. The useable storage space is contained between 142 meters and 128 meters elevation.

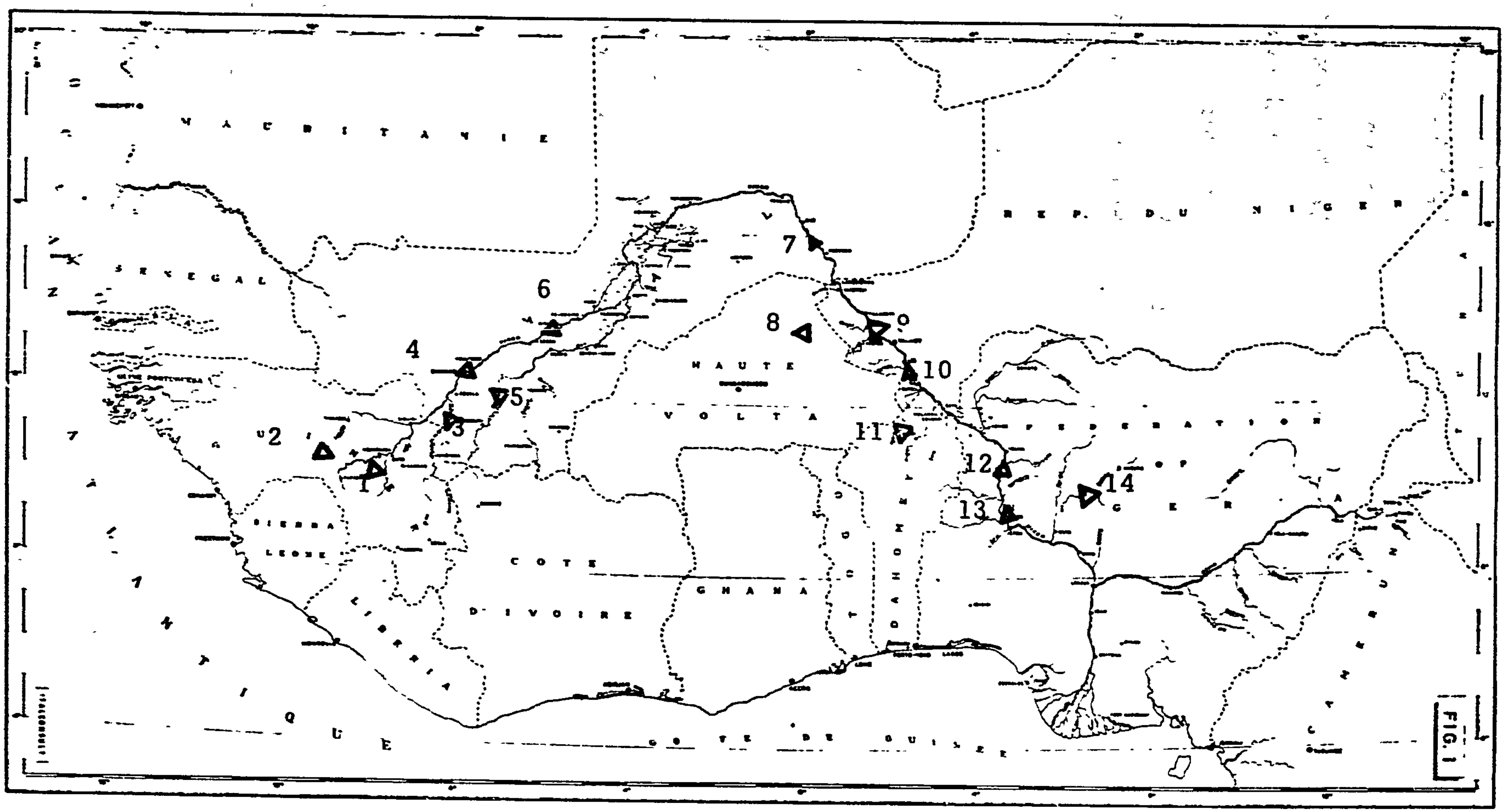


FIGURE 1.4-2 Existing and Proposed Dams and Hydroelectric Power Plants in Niger River Basin (continued)
(see key on following page)

KEY TO FIGURE 1.4-2

Number in Figure	Name	Comments
1	Fomi Dam	Proposed
2	Dabola Dam	Existing, Exact Location Unknown
3	Selingue Dam	Proposed
4	Sotuba Power Plant	Existing
5	Kamarato Dam	Proposed, Exact Location Unknown
6	Markala Power Plant	Existing
7	Ansongo Dam	Proposed
8	Beli Dam	Proposed, Exact Location Unknown
9	Kandadji Dam	Proposed, Exact Location Unknown
10	"W" Power Plant	Proposed
11	Mekrou Dam	Proposed, Exact Location Unknown
12	Kainji Dam	Existing
13	Jebba Dam	Proposed
14	Shiroro Dam	Proposed

The dam is operated to provide an average discharge downstream of 1500 m^3/sec year-round. The general operating policy is to store flood waters from September until October or November. Then, the natural flow is passed from October or November until March; and low flow augmentation takes place from March to July. Information is not available on the operation from July to November.

Presently, the power facilities associated with the dam have a capacity of 320 mw. The anticipated full capacity of 960 mw is to be installed by 1982.

The downstream discharge of 1500 m^3/sec makes year-round navigation possible between the ocean and Jebba for vessels drawing between four and five feet of water. Holmes and Narver (1968) report that since the depth of the Kpataki rapids has been increased, the Awuru rapids bypassed by a channel, and the Bajibo rapids blasted, year-round transportation is now possible up to Yelwa.

Average annual evaporation at the Kainji Dam site is approximately that at the Siguini since both have approximately the same latitude and elevation. The rate at Siguini is 1,611 mm. A pan coefficient of 0.6 is needed for adjustment. The annual average evaporation rate is 967 mm.

Since the average surface area of the reservoir is $1.250 \times 10^3 km^2$, the total annual evaporation loss is:

$$0.967 \text{ m} \times 1.250 \times 10^9 m^2 = 1.21 \times 10^9 m^3.$$

Monthly average is $0.1 \times 10^9 m^3$ or 38.4 m^3/sec. Therefore, the total annual inflow must equal at least:

$$1500 \ m^3/\text{sec} \times 365 \times 24 \times 3600 + 1.21 \times 10^9 \ m^3 = 48.5 \times 10^9 \ m^3.$$

4. Dabola Dam, Tinkisso River, Guinea.

This dam, begun in January, 1972, is for flood control and hydroelectric power (1500 kw installed capacity). The average discharge downstream at Tinkisso is 92 m^3/sec (Italconsult 1962).

Proposed dams and other projects — Holmes and Narver (1968) report that the potential for dam construction on the Niger River Basin is limited by highly variable seasonal discharges (requiring large reservoirs and expensive dams), the limited number of good sites, and the economic infeasibility of some physically good locations such as Ansongo on the Niger, which are far from demand centers.

Several dams are, however, being considered. The most promising are the Selingue Dam on the Sankarani River in Mali and the Fomi Dam on the Niandan River in Guinea. All known sites are shown in Figure 1.4-2.

1. Selingue Dam

In a recent study of this dam, Carlo Lotti and Co. (1972) decided that optimal multipurpose use is for power production and low flow augmentation, and that the "best" useful storage capacity of the reservoir is $1,928 \times 10^6 m^3$ (348.5 m level) (Total capacity = $2,166 \times 10^6 m^3$). Fourteen meters high, the dam would consist of earth embarkments with a concrete spillway. Other capacities considered were $1,512 \times 10^6 m^3$ (347.5 m level) and $1,049 \times 10^6 m^3$ (346.0 m level).

The average monthly flows entering Selingue and Koulikoro are shown in Table 1.4-11. They are based upon records to 1971. The average annual Koulikoro flows are less than those reported by Italconsult (1962) in Figure 1.3-6. Therefore, to develop a set of consistent figures, the inflows in Table 1.4-11 were adjusted so that the Koulikoro averages were the same. This was done by multiplying the 1971 values by 1798/1539, the ratio of the Italconsult (1962) Koulikoro average to the Carlo Lotti and Co. (1972) Koulikoro average. The adjusted Selingue flows are in Table 1.4-12. The minimum discharge maintained at the dam would be 150 m^3/sec (Holmes and Nariver 1968).

The River Niger Commission (1973) reports that the installed power capacity would be 11,200 Kw after completion, with potential

TABLE 1.4-11

Average Monthly Flows at Four Stations

Station	J	F	M	A	M	J	J	A	S	O	N	D	Mean
Dialakoro	329	183	96	69	107	382	1,074	2,440	3,922	3,292	1,540	653	1,181
Koulikoro	403	197	102	67	95	350	1,214	3,162	5,290	4,614	2,110	866	1,539
Kitango(aval)	355	155	80	52	61	215	1,007	2,761	4,732	4,510	2,177	858	1,420
Selingue	101	60	34	28	38	126	319	842	1,299	1,009	485	199	374

Source: Carlo Lotti and Co. 1972

TABLE 1.4-12

Corrected Flows at Selingue Site for Italconsult Figures

Month	Flow (m^3/sec)
J	117
F	70
M	40
A	33
M	44
J	147
J	373
A	984
S	1,518
O	1,179
N	508
D	232
Mean	437

expansion to 44,800 kw. The regulated flow would also increase power production at Sotuba and Markala and increase the navigable period between Koulikoro and Mopti by means of discharges in the dry season. Some navigation improvements would be needed to accompany these releases.

Flow regulation would also permit agricultural expansion along the Niger, bringing into production a total of about 55,400 ha of new land. More information is found in Carlo Lotti and Co., (1972). The dam will also produce water supply and fisheries benefits.

2. Fomi Dam on Niandan River (Guinea). Italconsult (1962) reports this dam would be used for navigation and agricultural improvements and power production. It would be an earth dam with a central concrete spillway ($4.5 \times 10^6 m^3$ of earth, $10^5 m^3$ of concrete). The reservoir's useful storage capacity would be $4.5 \times 10^9 m^3$. If we assume the total capacity is designed as in the Selinque Dam, it would equal:

$$\frac{\text{Total cap. Sel.}}{\text{Useful cap. Sel.}} \times \text{Useful cap. Fomi} = \frac{2.166 \times 10^9 \times 4.5 \times 10^9}{1.928 \times 10^9} =$$

5.053×10^9 m

The top water level would be 395 m elevation with a normal storage level of 371 m elevation. Maximum hydraulic head would be 44 m. The reservoir would fill from June 20 to December 20 and empty from December 1 to June 21, maintaining a constant discharge of 460 m^3/sec in the middle Niger. The installed generator capacity would be 85,000 kw. The average flow entering the site is 265 m^3/sec.

Regulation would make year-round navigation between Segou and Koulikoro possible without dredging work. Vessel draft size is unknown.

The dam's additional benefits at the Office du Niger would include: extending irrigation of Egyptian cotton through March 15; earlier sowing and pre-irrigation of rice; and growing two rice crops a year after a short-cycle corn crop.

3. Kamarato Dam on the Baoule Tributary (Guinea) is being considered for 1700 kw power.

4. Kandadji on Niger River (north of Niamey) is being considered for irrigation and power.

5. A dam on Beli River, a subtributary of the Niger in Upper Volta, is proposed to supply water for mining operations at Tambao and Tin Hrassan.

6. A power plant at the "W" on the Niger would be designed with 24,500 kw capacity. Heads utilized would be 10 m during low water (65 m^3/sec) and 5 m during high water (1,825 m^3/sec). The plant would not affect the regime of the river.

7. A power plant at Mekrou (at the confluence of the Mekrou and Tapou rivers) would have a large reservoir. The inflow at the site would average 10-15 m^3/sec in May-June and 150-400 m^3/sec in August-September.

8. Ansongo, Mali (Niger River), under consideration for navigation and power, is low priority since it is far from demand centers.

9. Jebba power plant, Nigeria, will be a river bed dam of rock fill with a central impervious core and some concrete works on the right side. The dam will have a useful storage capacity of 1.2×10^9 m^3 and a total of 3.1×10^9 m^3. Elevation will vary from 102.1 m to 97 m. As planned in 1962, 200 MW will be installed by 1984, 300 MW by 1988.

10. Shiroro power plant on the Kaduna will be a buttress type dam, with a useful capacity of 2.6×10^9 m^3 and total volume of

$3.2 \times 10^9\ m^3$. Water elevation will vary from 374.5 m to 344.6 m. Six 80 MW units will be installed: two in 1988, three in 1990, and one in 1992. Church (1970) reports that Shiroro will be used to generate power continuously when Kadura flows in June.

The flows entering the plant site will be concentrated in July-October (peak August-September), with minimum flows from December to May.

11. Holmes and Narver (1978) report that reduction of water losses in the delta may be possible if flow regulation and channelization of flow through swamps are used to reduce areas of shallow storage. However, since commitments and plans downstream have already been made based upon the present regime, they report the delta's storage efforts must be retained or replaced.

Cost of dams in Niger Basin — Costs of the Fomi, Selingue, and Shiroro dams can be determined as below.

As Carlo Lotti and Co. (1972) report, the Selingue Dam would cost \$23.6 million for the dam and associated structures, not including access routes, power plants, population resettlement, operation and maintenance, etc. Included in that cost are construction materials and design contingencies. If linearity is assumed, total cost of Fomi, a dam of similar construction, is:

$$\frac{5.053 \times 10^9}{2.166 \times 10^9} \times \$23.6 \text{ million} = \$55.06 \text{ million}$$

The Shiroro dam, like the Manatali dam, is to be a buttress-type dam. Manantali's 1969 estimated cost (Senegal-Consult 1970) was \$70.2 million, total capacity of $11.0 \times 10^9 m^3$. Comparing this price to the 1972 figures, let us assume a cost increase of 10 percent a year. The 1972 cost would therefore be:

$$(70.2)\ (1 + .10)^3 = \$93.4 \text{ million}$$

Again assuming linearity, the cost of the Shiroro would be:

$$\frac{3.2 \times 10^9}{11.0 \times 10} \times \$93.4 \text{ million} = \$27.2 \text{ million}$$

The costs of each dam, including only dams and associated structures, and design and contingencies, are summarized in Table 1.4-13. The annual operation; maintenance and replacement cost, are about .2 percent of the capital costs.

1.4.3 Lake Chad Basin

<u>Chad Basin Commission</u>: The Commission was established in 1964 and is charged with collecting and evaluating information on proposals made by member states, and recommending plans and joint research programs. The member states have also pledged to abstain from carrying out any work or scheme in the basin which would have an appreciable effect on the surface and groundwater flow in the basin. The member states are the four riparian countries.

TABLE 1.4-13

Cost of Major Dams in the Niger River Basin

Dam	Total Capacity	Construction	1972 Cost
Selinque	2.166×10^9	Earth embankment with concrete spillway	$23.6 million
Fomi	5.053×10^9	Similar to Selingue	$55.06 million
Shiroro	3.2×10^9	Buttress	$27.2 million

Navigation: There is local traffic on the lake itself. The Chari River is navigable between Sarh and N'Djamena from July to December. The river from N'Djamena to Lake Chad is navigable the year round and is much used for shipping. The Logone River is navigable only from N'Djamena to Moundou from August to October.

Fishing: ECA (1) (no date) reports that 100,000 tons of fish are caught annually in the Chari and Logone Rivers and Lake Chad. Dekker (no date) states that programs have been established to increase production and improve processing.

Possible Diversions, Canals, and Dams

Diversions and Canals

1. LCBC (1972) reports that flow diversion is possible from the Logone just above Bongor at Ere to Mayo Kebi and Benue River. A danger of this diversion lies in possibly upsetting the ecology of the Yaere flood plains as well as the irrigation schemes on El Beid. However, it was estimated that if flood flow in the Logone was sufficient to reach a peak of 1800 m^3/sec at Bongor, no danger would be done to the Yaere flood plain or El Beid irrigation. 1500 m^3/sec at Bongor would insure preservation of the fisheries and pasturage on the Yaere plain; an additional 300 m^3/sec is required to insure irrigation on El Beid. The average annual diversion from the Logone to Mayo Kebi and Benue River would be $.2 \times 10^9$ m^3 and would have no effect on the level of Lake Chad.

2. LCBC (1972) reports that a canal is possible between El Beid and Serbeouel River (just south of Lake Chad). Flood peaks occur at different times on these rivers and there is thus the opportunity to transfer flows between them so that each has a steadier regime (LCBC 1972).

3. LCBC (1972) reports that the channel of the Bahr-el-Ghazal could be improved to help regulate the level of Lake Chad.

Dams

There have been only a few dams or storage sites mentioned in the LCBC reports; none are on the Chari.

1. It has been proposed (LCBC 1972) that a series of dams be built on the headwaters of the Logone at Koumban on the Vina River, Goré on the Pende River, and on the Logone River itself at Moundou. These dams would limit the flood flow at Lai to a maximum of 1500 m^3/sec which would reduce the overspill at Lai. This would decrease water loss to Mayo Kebi and evaporation; increase the supply for Lake Chad, and make more water available for irrigation and navigation by flow release during the dry season. The dams would be of earth construction with concrete spillways and radial sluice gates. The gates would be closed in mid-August and opened gradually in October to provide flow not exceeding 1500 m^3/sec at Lai. The main purpose of the Moundou dam would be storage for very large floods. The Koumban Dam would store 5.0×10^9 m^3 and the Goré Dam would store 2.8×10^9 m^3.

2. It has been proposed to build a dam on the Mayo Kebi at Mbourao or Dao Koumi to store water for a proposed hydroelectric plant at Chutes Gauthiot near the Chad-Cameroun border. The dam

would be filled by the end of October with a volume of 10^9 m^3. This would provide enough water for a year. However, in a poor year, water would have to be withdrawn from the Logone during October (about $.9 \times 10^9$ m^3). If the Eré diversion were in effect, not enough flow would be provided at Bongor, and the Yaere Plain and the El Beid systems would be upset.

3. LCBC (1972) reports that the development of depression storage between Geidam and Gashua on Komadugu Yobe and between Geidam and the Lake is possible. They estimate that if sufficient sites could be found, 10,388,500 m^3 of water could be stored between Geidam and the Lake. LCBC (1972) says this would not effect the ecology of the Yobe Valley; no reason is given. They report the best way to operate the system would be to fill the depressions during the flood and then provide water for three months after the flood.

4. There is the possibility of storage on the Hadejia River in Nigeria. LCBC (1972) also recommends that more upstream storage sites be found on the seasonal rivers (El Beid and Komadugu Yobe).

Average monthly flows at diversions, canals, and dam sites are shown in Table 1.4-14. The source is LCBC (1972). The values at some sites were estimated by comparing their average annual flow values with the averages at sites where the complete hydrograph was known. There is no data available on the Hadejia.

Proposed and Present Irrigation Projects

LCBC (1972) describes the present and potential irrigation sites in the Basin. Sites are listed below: (I = present; F = proposed)

Cameroun

I Yagona

F Ft. Fourean and Logone Birni

F Maltam - Makari Section on Serbeonel

TABLE 1.4-14

Hydrographs at Diversion and Dam Sites (m^3/sec)

Month	Ere*	Touboro*,**	Gore*	Moundou	Mbourao	Geidam	Gashua
January	125	38	36	100	30	30	30
February	100	38	36	100	30	20	20
March	90	38	36	100	30	20	20
April	90	38	36	100	0	0	20
May	100	0	0	0	20	20	20
June	125	47	44	125	20	30	30
July	500	150	142	400	20	40	60
August	1100	375	355	1000	30	50	110
September	1750	525	497	1400	50	50	160
October	1800	375	355	1000	80	75	200
November	750	113	107	300	60	95	120
December	250	47	44	125	50	65	40
Yearly Vol. x 10^9 m^3	17.6	4.65	4.4	12.4	.73	.95	1.68
Mean Annual Flow	557	148	140	394	23.1	31.2	52.2

* Estimated

** Site of the Koumban dam is not known. Touboro is near the end of the river.

Chad

I	Yagona
I	Bol Dure
I, F	Sategui — Deressia Flood Plain
F	Mandelin (on Chari)

Niger

I	Diffa on Komadugu Yobe
I	Bosso — N'Guigmi Dune area
F	Plains between Geidam and Gashua and Ox-bow depressions between Geidam and Lake on Komadugu Yobe

Nigeria

I	Gamboru
I	Ngala (El Beid)
I	Yau, Daya, Abadan on Komadugu Yobe
I	Mallau Fortori
I	Maiduguri on Ngadda River
F	Between Geidam and Gashua and Geidam and Lake on Komagugu Yobe

Shown in Table 1.4-15 is a summary of sites where quantitative information is available.

Irrigation Potential From Lake Chad in AverageHydrologic Year

LCBC (1972) reports that by 2020 an area of 260,000 ha will be irrigated using water from Lake Chad sources. They estimate they will require 3.9×10^9 m^3 of water, or .5 liter/sec/ha. If we assume year round cropping at 1 liter/sec/ha, the total water need will be 7.8×10^9 m^3. It can be shown using a formula given in LCBC (1972) that this will lower the equilibrium level of the lake by 76 cm, approximately one meter.

Assume the average lake level is 4.13 m (at Bol) or 282 m.

TABLE 1.4-15

IRRIGATION IN CHAD BASIN

Location	Present	Future	Comments
CAMEROUN			
Yagona	3000 ha (controlled flooding)	Possible expansion	Drainage to Mayo Guerlo
	5000 ha (seasonal)		
CHAD			
Yagona	2500 ha (controlled flooding) 500 (yr. rd.)	Possible expansion	
Bol Dune	4000 ha (reclaimed polders)		Water from Phraetic wells
NIGER			
Diffa	275 ha (controlled flooding)	Small expansion	
NIGERIA			
Gamboru	1000 ha	Fully developed	Water from El Beid (seasonal)
Between Ngala and Lake	400 ha	16,000	Yr. rd. from lake
Komadugu Yobe (Yau, Daya, Abadan)	1600 ha	20,000	All from K. Yobe one crop in wet season
Mallan	50 ha		From lake
Fatori, Jerre Bawl	1500 ha		From Ngadda River

Since the 1971 level was 280 m and the lake has split apart, assume 281 m is the lowest level the lake can safely have. Therefore, the year round irrigation potential from Lake Chad is 260,000 ha in an average year. The potential from the rest of the area totals about 18,000 ha, most of which is seasonal. See Summary Table 1.4-16.

Potential With Drought

The potential for irrigation in a series of drought years is non-existent. As has occurred now, the lake has split apart and is not suitable for supplying irrigation water. In a drought, it is expected that most of the seasonal agriculture would be eliminated.

Regulation

There appear to be two storage-regulation schemes that are possible in the next 20-30 years. They are the storage of water between Geidam and the Lake on the Komadugu Yobe and the construction of headwater dams on the Vina, Pende, and Logone.

Storage at Geidam on Komadugu Yobe would provide storage for $10 \times 10^6\ m^3$ of water for three months after the flood. Since less than 10 percent of the runoff from this basin presently reaches the lake, and this system provides so little of the input to the lake, this storage would have no effect on the level of the Lake and it is expected it would not upset the Komadugu Yobe's ecology. If an average depression depth of three meters is assumed, this implies the surface area covered would be $3.3 \times 10^6\ m^2$. Using the evaporation rate from the lake, this implies a three month evaporation loss of

$$\frac{3}{12} \cdot 2.15\ m\ (3.3 \times 10^6\ m^2) = 1.7 \times 10^6\ m^3$$

Therefore, the approximate net amount available for crops would be $8 \times 10^6\ m^3$. At a consumptive use rate of $10^{-3} m^3$/sec/ha for three months, this implies the irrigation of

TABLE 1.4-16

SUMMARY OF IRRIGATION POTENTIAL IN LAKE CHAD BASIN

	Average Year	Drought (Ave. 1970-72 Conditions)
No Regulation	260,000 ha (year round) 18,000 ha (seasonal)	0?
Regulation	276,000 ha (year round) 18,000 ha (seasonal) 995 ha (3 months)	
Channelization (no upstream dams) (may not be economically or physically feasible, large estimate	400,000 ha (year round) 18,000 ha (seasonal) 995 (3 months)	70,000 ha (year round)

$$\frac{8 \times 10^{6} \text{ m}^{3}}{3 \times 31 \times 24 \times 3600 \times 10^{-3}} = 995 \text{ ha}$$

for three months. This is under average conditions of rainfall.

It is difficult to analyze the effects of the three headwater dams in detail as little information is available. However, LCBC (1969) reports that the annual loss of water between Lai and Bongor on the Logone River is $.9 \times 10^{9}$ m^{3}. However, only some of this leaves the basin (via the Mayo Kebbi). The average annual flow at Mbourao on the Mayo Kebbi is 0.73×10^{9} m^{3} (LCBC 1972). Tributary inflow from outside the basin is estimated to be about 0.5×10^{9} m^{3} annually. Therefore, the contribution from the Logone is 0.23×10^{9} m^{3} annually; hence, of the 0.9×10^{9} m^{3} that spills between Lai and Bongor, 0.23×10^{9} m^{3} is actually lost to the basin. Since the total volume of the Koumban and Gore Dams is 7.8×10^{9} m^{3}, this volume could certainly be stored upstream. If it is assumed it would be stored for a period of two months and that the evaporation loss is similar to that at Kainji the two month evaporation loss would be (assuming surface area is proportional to volume)

$$\frac{7.8 \times 10^{9}}{\underset{\text{vol. of Kainji}}{11.3 \times 10^{9}}} \times \underset{\text{Kainji}}{\underset{\text{evaporation loss}}{.1 \times 10^{9} \text{ m}^{3}/\text{month} \times 2 \text{ months}}}$$

$$= .138 \times 10^{9} \text{ m}^{3}$$

Therefore, the loss in storage would be 0.14×10^{9} m^{3} and the total amount of water saved would be 0.23×10^{9} m^{3} $-$ 0.14×10^{9} m^{3} $= 0.1 \times 10^{9}$ m^{3}.

If it is assumed that 50 percent of this reaches the Lake and is available for year round irrigation, this implies

$$\frac{50 \times 10^{6}\ m^{3}}{365 \times 24 \times 3600 \times 10^{-3} m^{3}/sec/Ha} = 1600 \text{ hectares}$$

It should be noted that this storage will probably keep the peak at Bongor below 1800 m^3/sec and prevent sufficient water from spilling over the Yaere flood plain and El Beid. Therefore, to prevent upsetting the ecology of the flood plain and the irrigation schemes on the El Beid, water will probably have to be pumped.

Regulation Under Drought Conditions

It is expected that under drought conditions the depressions on Komadugu Yobe will not fill and no irrigation will be possible.

It also appears that the headwater dams will not be helpful during drought periods if their only use is to prevent overflows at Lai and Bongor during the flood season. During a drought such overflows would probably not occur. Therefore, no water would be saved by the dams. The operating policy for the dams during a drought would probably be to let all water pass and flow to Lake Chad to prevent as much as possible the lake level from falling. Irrigation water probably should not be withdrawn.

Channellization

There is no mention in the LCBC reports of the feasibility of major channellization of the Logone and Chari to prevent losses from infiltration and spills. There does not appear to be a large potential for upstream storage. Therefore, most of the water would have to be stored in the lake. Levees would have to be built around the lake. If the approximately $5.3 \times 10^{9}\ m^{3}$ spilled, and lost, between Lai and Miltou and N'Djamena were available, the agricultural potential of the area would be greatly increased. Over a year, this would provide irrigation for 140,000 additional hectares.* The dams in this case would not be needed. In drought conditions, the total amount of irrigation possible would be about 70,000 ha.

Cost of Headwater Dams

The cost of the dams at Gore and Koumban can be estimated since their volumes are given. They are earth dams with concrete spillways like the Selinque Dam. Based upon the cost of the Selingue Dam, the costs of the dams, and their associated structure and design and contingencies (no operation, maintenance, or repair costs) were calculated, and are shown in Table 1.4-17.

1.4.4. Summary of Water Availability for the Three Major Basins

SENEGAL BASIN

The flow volumes reported in this section could be divided between the riparian areas of Senegal, Mauritania, and Mali since the waters originate either in or above Mali, and Senegal and Mauritania are downstream from Mali. The navigation estimates are given only for Senegal and Mauritania because no data exist for Mali.

Average Conditions

Navigation and salinity control at the Delta require a flow of 115 m^3/sec at Bakel, Senegal. Irrigation is therefore possible between July and December . Navigation would be possible between June and January.** The total volume of flow in this irrigation period (minus 115 m^3/sec for navigation) is 22×10^9 m^3.

The maximum amount of regulated flow that could be provided is 700 m^3/sec. This would be the year round monthly average at Bakel.

*This assumes that lake loss via evaporation and infiltration remains the same -- i.e., it does not change significantly because of the larger average surface area of Lake Chad.

**1.4 m draft boats, St. Louis to Ambidedi

TABLE 1.4-17

Costs of Headwater Dams in Chad Basin

	Estimated useful volume	1972 cost
Koumban	$5 \times 10^9\ m^3$	\$61.2 million
Gore	$2.8 \times 10^9\ m^3$	\$34.3 million

Allowing for navigation, this leaves 585 m^3/sec for irrigation or 18.4 x 10^9 m^3 (January - December).

Drought Conditions

It is assumed that the conditions at Bakel during the 1913 drought are similar to average 1970 - 1972 conditions. Without regulation, and allowing 115 m^3/sec for navigation and salinity control, this provides for navigation between June and December and irrigation between July and November using a volume of 6.0 x 10^9 m^3.

With regulation, the average monthly drought flow at Bakel would be 390 m^3/sec. Therefore, both year round navigation and irrigation could be provided. The volume available for navigation would be 8.7 x 10^9 m^3.

NIGER BASIN

Average Conditions

In average years, the average flow at Kainji Dam is 2110 m^3/sec. Since about 1200 m^3/sec is from tributary inflow south of the Niger border (i.e., it either originates in Dahomey or Nigeria), about 910 m^3/sec is from Niger. Since about 50% of this volume has been lost in passage through the Delta, the flow entering the Delta is 2 x 910 or 1820 m^3/sec or 59 x 10^9 m^3 annually.

A volume of 50 x 10^9 m^3 (1540 m^3/sec) is needed for Kainji Dam. Since the yearly input from tributaries is 38 x 10^9 m^3 (1200 m^3/sec), about 12 x 10^9 m^3 is needed from Niger. Therefore, at least 24 x 10^9 m^3 must enter the Delta. Assume 15 x 10^9 m^3 is used before the Delta. Therefore ((59-15)/2) 22 x 10^9 m^3 is available after the Delta. However 12 x 10^9 m^3 must be reserved for Kainji. Therefore 10 x 10^9 m^3 is available for use in the rest of Mali and Niger.

With regulation and Delta improvement (i.e., decreasing Delta loss from 50 percent to 25 percent), it can be shown in a similar manner that if 30×10^9 m^3 is used above and in the Delta, the flow available to the rest of Mali and Niger would be 8×10^9 m^3.

Drought Conditions

Under drought conditions, flows are reduced by about 44 percent.

If it is assumed that Kainji Dam will cut back its power production and use 30×10^9 m^3 annually instead of 50×10^9 m^3 and since the tributary inflow is now 21×10^9 m^3, 9×10^9 m^3 is now needed from Niger. Since the flows entering the Delta are now about 33×10^9 m^3 if 10×10^9 m^3 is allocated above the Delta 3×10^9 m^3 would be left for use in the rest of Mali and Niger.

A similar analysis of regulated conditions shows that if 10×10^9 m^3 is used before the Delta, 7×10^9 m^3 will be available for the rest of Mali and Niger.

Navigation

Under all these conditions, there would be navigation possible for approximately one meter draft vessels between Koulikoro and southern Niger. The navigation periods would range from two to seven months.

CHAD BASIN

Presently, there is little irrigation in the Basin. Given below is the total volume that could be withdrawn from the lake. For purposes below, it is assumed that the maximum amount of permanent lowering of the lake level is one meter. Since over 95% of the water entering the lake enters from the Chad side (Logone/Chari system), the volume given could either be entirely allocated to the riparian areas in Chad south of the lake or else distributed between Chad and Niger (assuming Niger takes from the lake).

Average Conditions

In an average year, the volume that could be withdrawn from the lake is $7.8 \times 10^9 m^3$ (no regulization or channalization). While channelization of the Logone/Chari system to reduce overflow losses has not been suggested in the literature, it may be feasible. If done, this would provide an additional $5.3 \times 10^9 m^3$ of water. The total available would be $13.1 \times 10^9 m^3$.

Drought Conditions

As has occurred now, the lake would split apart. Therefore, with drought, no large water withdrawals are possible from the lake (or the Lagone/Chari system) under natural conditions. With channelization, a volume of approximate $2.6 \times 10^9 m^3$ would be available under drought conditions, (most of it between July and November, the wet season.)

1.4.5. Other River Systems

Volta River

The Akosombo Dam (Ghana) is 110 km from the sea. The surface area of the lake formed by the dam is $8476 m^2$ and it holds $147.6 \times 10^9 m^3$. The lake supports various industries including fishing and agriculture, and is extensively used for navigation. The hydroelectric power station has a generating capacity of 750,000 kw. Most of the power is consumed by industries in the Accra-Tema region, Ghanas's major industrial area (Grove 1967).

The following section is from Dekker (no date):

"The development of the upper part of the Volta River will demand co-operation between Upper Volta, Ivory Coast and Ghana. The urgent problem to be solved is that of the control of onchoceriasis (river blindness). Fortunately, steps are being taken to arrive at a

West African regional program of control of the vector of this disease. However, settlement should follow control. The center south of Upper Volta and the North of Ghana and Ivory Coast are at present relatively thinly populated, while certain areas of Upper Volta suffer from over-population with respect to land resources. The area is suitable for the construction of small barrages for cattle watering and small scale irrigation. If tse-tse control could be effectively undertaken, valuable grazing land would become available. Ghana and Ivory Coast are natural markets for cattle from Upper Volta. Recently, agreements to promote cattle export from the latter country to Ghana have been concluded. There are also proposals for using the large man-made lake of the Volta for transport in this interstate trade. (Church 1970) reports that in general, the 200 mile long waterway of the lake and associated ports and barge traffic should greatly help Upper Volta and Mali.)

Upper Volta is particularly poor in energy resources. One way of coping with the projected power demand in 1975, would be to install additional thermal power of approximately 6000 kw in 1970 and 10,000 kw in 1975. However, there is a possibility of constructing a hydro-electric plant in Ghana not far from the border near Lawra on the Black Volta with an installed capacity of 36,000 kw. If favorable loan conditions could be obtained, it appears possible to arrive at a price of energy at the terminals of the transformers in Upper Volta which is about one half of the costs of energy produced at thermal plants in that country.

"It is not certain that Ivory Coast will be able to construct on time the hydro plants of Kossou and Attakro necessary to cope with the demands during 1975-1980. There is the possibility of connecting the Akosombo plant with Ayame in Ivory Coast, a connecting high tension line of 120 km being the main requirement. Also, the construction of a reservoir on the Bia river in Ghana could increase the production of energy at the Ayame plant, situated on the same river in Ivory Coast. There are three more potential power sites identified in Ghana not far from the border with Ivory Coast. The distance between Lagos and Akosombo is less than 350 km. Thus, it would be possible to connect the plants of Kainji (and the projected additional hydro plants of Jebba and Shiroro near Kainji) with Akosombo and, further, with Ayame and Abidjan. The distance between Lagos and Abidjan is 840 km which corresponds with a shift of about half an hour in time of daylight. This will have a slight but favorable effect on the combined daily demand curve."

GAMBIA BASIN

(This section is from Dekker (no date)):

"The Gambia basin is another West African river basin of which the resources can only be developed through a co-ordinated effect by the three Governments concerned: Gambia, Senegal, and Guinea.

"The Gambia river has the largest estuary of Africa and is navigable for seagoing ships for more than 200 km. Yet, due to the fact that Gambia was in the past administered by the United Kingdom and its hinterland by France, no major seaport has come into existence. Flood protection and irrigation could substantially increase the agricultural productivity of the lower part of the basin. In the upper parts of the basin there are possibilities for storage and an important potential for hydropower (estimated at 900 million kwh/annum).

"The governments of Gambia and Senegal have created an interstate committee for the development of this basin. A survey of its resources and preparation of a development plan is being undertaken with assistance from the UNDP."

1.5 Groundwater

Theoretically, to evaluate the supply potential of groundwater systems, the minimum information needed is:

1. Where the different aquifers and their inter-connections are.
2. The elevations of their piezometric surfaces.
3. The depth and thicknesses of the aquifers.
4. The coefficients of storage and transmissibility of the aquifers.
5. The recharge mechanisms of the aquifers.
6. The soil and vegetation types covering the aquifers.
7. The geological composition of the aquifers.

The sources of this information are:

1. Geological Surveys.
2. Exploratory Drilling and Well Logs.
3. Well Pump Tests.
4. Records of the Fluctuations of Existing Wells.
5. Basic Hydrological Streamflow and Rainfall Data.

From a review of several general reports on groundwater in West Africa as well as all available groundwater publications on Mauritania and Senegal, it appears that the following is known about West Africa aquifers:

1. Their general location.
2. The elevations of the piezometric surfaces of at least the phreatic aquifers and in some cases, the deeper artesian aquifers.
3. The depth and thicknesses of at least the phreatic aquifers and some of the deeper artesian aquifers. However, much of this information is based upon a few geological cuts and may not be reliable. In a few areas, such as south central Mauritania, there are no geological cuts. No one knows the thickness of the Maestrichtian aquifer in this part of Mauritania.

4. The coefficients of storage and transmissibility are known at only a few specific locations. Generally, these coefficients vary from location to location in the large aquifer. However, if enough pump tests are done and field experience is available, good overall values can be determined.

5. The recharges of only some of the aquifers have been studied. Generally, studies have been qualitative, with broad estimates.

6. The soil and vegetation covering the aquifers are generally known, as are the approximate geological compositions of the aquifers.

7. The quality of the well inventories and records vary by country. For example, LaRue (1974) reports that the government of Niger has catalogued information on 13,000 of the 20,000 wells in the Country. They reportedly have information on the yields and draw down at each location. If such information is available for several years at most wells, it would be extremely valuable in determining the development potential of the corresponding aquifers. However, Lakh (1974) reports that the government of Senegal only has information on the locations of the deep wells (forages) in the country that service Dakar, the villages, and some of the factories.

A summary of the inventory and research activities that have been done in the Sahel-Sudan nations is in U.N. (1973).

Based upon this information (much of it gathered during staff visits to obtain data in Africa and France), it appears that one can only estimate very generally the overall quantities and costs of groundwater development in West Africa, even if one had all the existing reports readily available. Therefore, rather than devoting substantial and unavailable resources to obtaining and evaluating all reports in detail, (especially since BRGM has a contract with France to do this), the water resources staff has concentrated upon gathering groundwater data that is in summary form, and on developing data that is representative of the actual situation in the Sahel-Sudan zone. This data was developed by a review of the available groundwater literature on Mauritania, and in consultation with Cederstrom (1974). The information was then used in the supply model discussed in Part II.

1.5.1 <u>Principle aquifers</u>: African Shield: This consists of crystalline and metamorphic pre-Cambian formations. As seen in Figure 1.5-1, there are outcrops of the shield throughout the study area, particularly in Upper Volta. Groundwater is found in weathered and fractured zones of these outcrops (the formations themselves are impermeable). These groundwater reservoirs are generally local in extent. The groundwater level may lie near the surface and be subject to evaporation, or it may be as much as 100 to 200 meters below the surface. Some of this water may be fossil water. Fossil water is water that is not being recharged. Infra-Cambian cover overlying the "shield" generally is a mediocre aquifer in west Africa.

<u>Sedimentary Basins</u>: Large sedimentary basins exist in the study area (see Figure 1.5-1). Large amounts of groundwater generally exist in these basins.

One of the deeper basins is the "Continental Intercalaire" (Nubian Sandstone). It is of the lower Cretaceous Age and chiefly sandstones and clays. It is believed to be a large source of fossil water (thousands of cubic kilometers). Its great depth (greater than 1000 meters in many cases) may make it difficult to obtain.

The "Continental Terminal" is of the upper Cretaceous and lower Eocene age. It is less extensive than the Continental Intercalaire and is interrupted by non-aquifer formations. However, its depth is less and hence it is more accessible.

The other types of sedimentary aquifer formations date from the Pleistocene and recent epochs, and include sand dunes, river beds, coastal deposits, and other surface formations.

1.5.2 <u>Recharge</u>: There are two types of recharge: direct and indirect recharge. Direct recharge occurs when rainfall moves

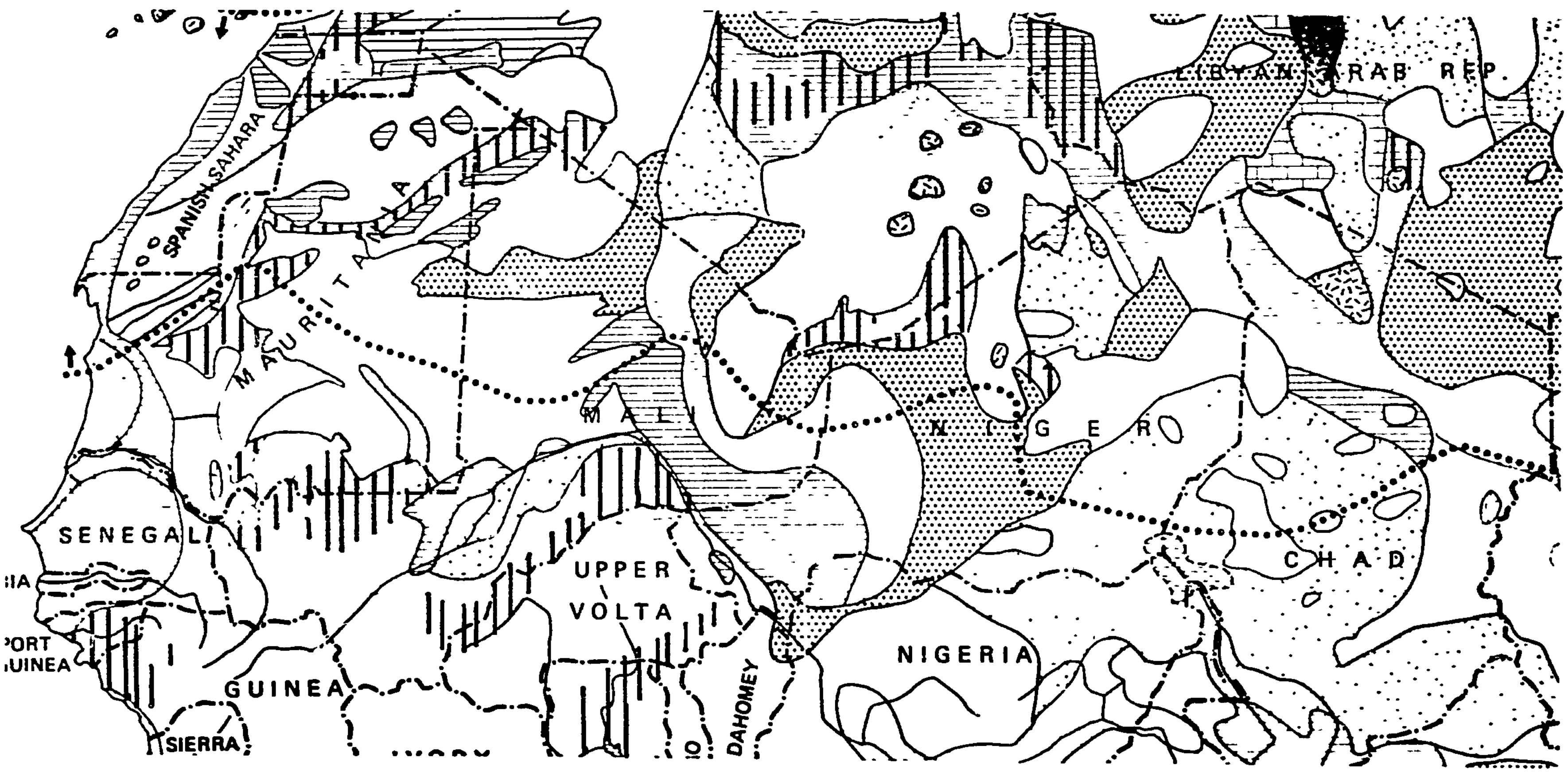


FIGURE 1.5-1 Major Aquifers in West Africa

Source: U.N. 1973.

(see key on following page)

AQUIFERS WITH INTERSTITIAL POROSITY

Extensive sandy areas (Sahara)

Alluvial fills, deltas, chott deposits
Quaternary formations of the Chad and Congo basins
Coastal sedimentary basins

Sandstones, conglomerates, etc. of the *Continental terminal*
Kalahari sandstones and sands (south of the equator)

Continental intercalaire, Nubian Sandstones, sandstone Karroo
and other Pre-Cretaceous or Cretaceous continental sandstones

AQUIFERS WITH FRACTURE AND CHANNEL POROSITY

Limestone platforms of the hamadas of northern Africa (Plio-Pleistocene)

Karstified limestone aquifers: Jurassic, Cretaceous, Eocene

Complex-structured tectonized zones of northern Africa
Marly sandstones, marly limestones, flysch, etc. of the Jurassic and Cretaceous

Calcareo-dolomitic massifs and plateaux of the Upper Pre-Cambrian
and the Cambrian

FORMATIONS WITH LITTLE OR NO POROSITY EXCEPT LOCALLY IN CERTAIN FAVOURABLY SITUATED WEATHERED OR FISSURED ZONES

Hard sandstones, shaly sandstones, quartzites: Pre-Cambrian and Palaeozoic

Infra-Cambrian (Primary) schists of the Karroo
Predominantly Eocene clays

Crystalline and metamorphic rocks

Eruptive volcanic rocks

Annual rainfall less than 100 mm

Key to Figure 1.5-1

Source: U.N. 1973.

directly into an aquifer. Slatyer and Mabbutt (1964) report that the most important factor for direct recharge in arid areas is the nature of the ground surface and the subsurface layer; both must be highly permeable. Another major factor affecting direct recharge is evapotranspiration rate. A general rule of thumb in the area is that in regions where annual rainfall is less than 300 mm (the value of 400 mm has also been used), no direct recharge takes place because of evapotranspiration. Bourguet (1966) notes some exceptions to this, e.g., in sandy areas where rainfall is particularly intense or greater than normal.

Indirect recharge is recharge that occurs when water is concentrated by streamflow or sheet flow. (Both will be referred to as runoff.) Runoff depends upon duration and intensity of rainfall, vegetation, permeability and slope. Slatyer and Mabbutt (1964) report that the last 3 factors are the most important.

To develop recharge values that are representative for the area, the aquifers in the region were divided into three general classes: sedimentary formations with large storage capacities such as the Continental Terminal; sedimentary formations with low storage capacities such as ephemeral surface aquifers; and rock aquifers (those of the African Shield).

Sedimentary Formations with Large Storage Capacities

Generally, these sedimentary formations are overlaid by a fairly permeable surface layer from which there is little, if any, runoff. Therefore recharge must generally be direct. Only areas where the average annual rainfall exceeds 300 mm are considered to be recharged directly. The amount of recharge is assumed to be a percentage of the rainfall. The percentages used are:

Annual Rainfall(mm)	Annual Percentage Recharge
300 - 500	7%
over 500	12%

These percentages are one half those used to calculate recharge in the Las Vegas basin of Nevada (reported in Farvolden 1968). The values are halved because the Nevada rainfall is more frequent.

Another form of recharge to these aquifers is from a bordering area of high runoff. This is the situation in Mauritania where runoff for Precambian bedrock in the south-central area recharges the Western sedimentary basin (Paloc 1962). It is assumed that runoff forms only if the monthly rainfall exceeds 15 mm. The runoff coefficient is assumed to be 10 percent (typical runoff coefficient for Mauritania) (Bourguet 1966). 50 percent of the runoff is considered to be recharged. This is the value used by several researchers investigating groundwater recharge in a piedmont arroyo (Slatyer and Mabbut 1964).

Sedimentary Formation with Low Storage Capacities

It is assumed that 5 percent of the area is covered by ephemeral aquifers. These are superficial sand and alluvial aquifers that are not very thick and are subject to high evaporation losses. They are presently usually exploited by small dug wells. They are recharged by both runoff and rainfall. In areas where the monthly rainfall exceeds 15 mm, the recharge rate is assumed to be 20 percent of such monthly rainfall. The recharge to and baseflow from sedimentary aquifers bordering rivers and hydraulically interacting with them has to be determined from available data.

Rock Aquifers

Recharge to rock aquifers is mainly from the collection and infiltration of runoff. It is assumed that runoff forms only if the monthly precipitation exceeds 15 mm. The runoff coefficient used is 10 percent. 10 percent is assumed to infiltrate (net is 1 percent) If the annual precipitation exceeds 300 mm, an additional 1 percent of the monthly rainfall (whether it exceeds 15 mm or not) is also considered recharge.

1.5.3. Lift Heights, Well Depths, Yields, Distances Between Wells

These values were based upon Cederstrom's (1974) review of the summary of data for Mauritania and West Africa, and his knowledge and experience. They are all reasonable yet conservative estimates. The yields are slightly greater than the values he recommended. The values for the phreatic Continental Terminal and Rock aquifers are when they are recharged. If not recharged, the values of the well depths are used for the lift, and the yields are decreased by 50 percent.

It is assumed that all wells are fully penetrating, and that all wells are operated 10 hours per day, 75 percent of the year, to allow for partial recovery of the aquifer.

The distance between wells is based upon several considerations. In aquifers where there is no recharge, it is advisable to keep the wells far apart to maintain the piezometric head. In recharge areas, wells can be kept fairly close; in fact, closely spaced wells in proper locations may intercept flow that would otherwise leave the basin.

The values used are in Table 1.5-1. They are for recharged formations.

TABLE 1.5-1

Aquifer Yields and Characteristics

Aquifer Type	Lift(m)	Well Depth(m)	Yield (m^3/hr.)	Distance between wells*
Sedimentary, Large storage capacity				
Sand Dune	40	60	6	250 ft. (1000 ft.)
Phreatic Continental Terminal or Similar	30	40	25	250 ft. (1000 ft.)
Intermediate Artesian Maestrichtian sand or similar	30	150	35	(3 miles)
Continental Intercalaire (Nubian Sandstone)	0 (fully artesian)	2000	35	(3 miles)
Sedimentary, Low storage capacity				
Ephemeral	3	4	5m^3/day	close together
Alluvial Bordering a River	3	4	15	close together
Rock	30	50	1	500 ft.

*Values in ( ) are for no recharge conditions.

1.5.4 Cost of Groundwater

In Table 1.5-2 are estimates of the costs of installing wells of the types specified for the yields shown. The estimates follow the methods used in U.S.G.S. Water Supply Paper 2034, "Cost Analysis of Ground-Water Supplies in the North Atlantic Region, 1970" (Cederstrom 1973). This paper contains the first large-scale estimates of cost of yield for groundwater appropriate for mathematical programming models of the type used in this study. The estimating methods were adjusted for Sahel-Sudan characteristics with the assistance of Dr. C. J. Cederstrom, the author of the paper.

To the costs of installation shown in the table, 2 percent of installation costs should be added for maintenance on a yearly basis, and $4.10 per 3785 liters (1000 gals.) for energy costs where gasoline driven pumps are used. U.S. costs were increased by 30 percent to account for additional transportation and maintenance costs in west Africa.

The four types of wells for which estimates are made by Cederstrom are

1) Shallow dug wells, concrete lining. Subject to seasonal recharge and failure in long dry periods. Approximately 4 m deep.

2) Ephemeral, alluvial aquifers. Water table water at no great depth, max 40 m -- generally subject to recharge and reasonably dependable where rainfall is greater than 3 m. Small diameter. (Phreatic Continental Terminal or similar).

3) Wells in consolidated igneous and metamorphic rocks. Low yields. Depths 50 m (may be more where rock appears favorable). Small diameter.

4) Wells in semi-consolidated porous sandstone, water under pressure head, nominal diameter. (Intermediate Artesian).

The costs of wells in the other formations were developed by the project staff using this data.

TABLE 1. 5-2

Cost of Groundwater

1. Shallow Dug Wells

Culvert	$200	US 1970
Labor	10	
Miscellaneous	40	
	$ 250	US 1970

NOTE: These wells are probably hand dug. Possible depth 4 m.

2. Water Table Wells

Drill Plus Casing	$1200	US 1968
Screen	150	
Development	100	
Pump	750	
Test	——	
House	50	
Engineering and Contingencies	500	
TOTAL	$2750	
Adjust for 1970 (+ 10%)	275	
TOTAL	$2925	US 1970

NOTE: The casing on such a well might be stainless steel.

3. Rock Wells

Drilling	$2300	US 1968
Pump	750	
Development	———	
Test Run	100	
House	50	
Engineering and Contingencies	800	
TOTAL	$4000	
Adjust for 1970 (+10%)	400	
TOTAL	$4400	US 1970

4. Wells in Semi Consolidated Porous Sandstone, Pressure Head

Drilling	$7650	US 1968
Screen	800	
Development & Casing Cementing as required	400	
Pump	3500	
Test	200	
House	100	
Engineering and Contingencies	3230	
TOTAL	$15930	
Adjust for 1970 (+10%)	1593	
TOTAL	$17523	US 1970

1.5.5. Indigenous Methods: Choice of Technique*

Some projects are established largely as if they were to be performed in a technologically developed country. It is a mistake to not inquire about and borrow from other developing countries, devices put into practical use by indigenous peoples. As an example, in the matter of shallow wells cased with concrete culverts - a mechanical winch on a truck is not necessary to handle these fairly heavy cumbersome units - a tripod of palm tree logs can be created, and with a rope and simple pulley these units can be manhandled into place.

In the Fezzan (Libya), somewhat shallow drilling into the Nubian sandstone is accomplished by a simple tripod. A rope over a pulley or sheave suspends the bit in the hole, the far end of which is snubbed on a "dead man". To activate the bit, 3 to 5 men pull on the "fore" rope (raising the bit) and then release the rope (dropping the bit). A string of .15 m pipe is also managed ingeniously. A string of 9 to 45 meters of pipe is heavy indeed. Assuming that the hole is nearly full of water, a length of pipe is lowered in the hole - it does not sink because its lower end is plugged into a wooden plug and therefore tends to float. A new length is added at the surface and the string of pipe lowered that much. Bouyancy is gained from the hollow new pipe added, and the string sinks slowly. When the full length of casing is in place, the plug is drilled out. The method is suitable where the casing can rest on a hard stratum, through which an open hole is continued to an aquifer below. In the Fezzan, wells up to 100 meters deep have been constructed using the method of drilling and casing outlined.

To set a screen below the casing would be difficult but might be accomplished if sufficient ingenuity were brought to bear. Where

*This and the following section were contributed by Dr. C. J. Cederstrom (1974) the final casing is of small diameter, it might not be difficult to lower a total string of screen and casing into the hole (no plug in this string) by a tripod device and surface clamps. Drilling out a plug in a .05 m line might be found to be troublesome.

Insofar as pumping devices on small diameter wells are concerned, a pump jack activated by a gasoline engine may be a proper technological answer. However, where lift is small, a hand pump should be sufficient; where lift is great, we might visualize a pump jack, not activated by a gasoline engine, but rather, a large wheel on either side of the jack with handles which when turned by hand would permit activation of the jack and in turn the up-and-down motion of the pump rods.

Where very great lifts are concerned and volume discharge is moderate to large (over perhaps 100 lit. /min.) we recommend the helical positive displacement pump rather than conventional turbine, submersible turbine, or any other type of pump. Casing size may limit the use of such pumps.

Jet pumps are very tricky to install, regulate or repair. They should not be considered except where skilled technicians are at hand to care for them.

Windmills may be practicable in many areas. The volume is generally small, perhaps up to 40 lit. /min. and unless guyed additionally are subject to being blown over by very high winds. They must not be allowed to run at high wind velocities, thus requiring some attention. Two-vane fans are said to be as efficient as conventional many-vaned fans and would be less subject to wind destruction if shut off.

Utilization of any of these or comparable back-country devices may save considerable money in original installations of thousands of water point wells for cattle or small villages, further savings in operation, and lower maintenance costs. Completion of somewhat primitively constructed wells will be slow, but may not require as much time as assembling and putting into operation equipment used in technologically advanced areas. Further, continued operation of hand operated pumps, for instance, will be easier to accomplish than comparable operation of gasoline operated pumps.

1.5.6. Quality of Water

There is ample reason to believe that corrosive ground water will be encountered in much of the desert to semi-desert country. To insure a reasonable life of wells, ordinary steel pipe will not do. Either fiberglass or stainless steel casing must be used. It is a fact that thin wall stainless steel casing is equal or better in strength than fiberglass casing. Further, connections on a string of pipe - perhaps more than 100 meters - can be made by positive welds rather than by gluing as with fiberglass casing.

1.5.7. Water Potability

Groundwater in the area is generally of good quality, suitable for both human and animal consumption. However, there are some areas where the groundwater is brackish -- both inland and near the coast. Since only limited data are available, generally all aquifers with a few exceptions can be considered potable.

1.5.8. Groundwater Use

Groundwater is widely used throughout the study area. Wells vary from essentially hand dug holes into ephemeral aquifers, to tubewells supplied with modern pumps. Groundwater is used for livestock watering, industrial water supply, and human needs. In fact, it is the major source of water in population centers not having readily accessible surface sources (i.e., there is little long range surface water transmission - the one case probably being the St. Louis-Dakar pipeline). Groundwater is particularly favorable for human consumption because it is generally purer than surface water.

To the authors' knowledge, groundwater is not presently being widely used for irrigation in the study area. However, it has been proposed. For example, the President of Niger has requested $250 million to build 2500 wells at an average depth of 900 feet to irrigate 6 million acres (Ottoway 1973).

U.N. (1973) summarizes much of the recent and proposed groundwater work in each country.

1.6 Problems Accompanying Water Resource Development In the Region

The benefits of water resource development are obvious. Irrigation will increase agricultural output. Dams can provide an economical source of electrical energy which can boost an economy and more dependable water supply. Deeper low flows can improve navigability and hence increase trade and transportation. Floods can be controlled. Dependable domestic water supplies can be established, etc. However, there are many problems that may arise from water resource development in the area. These must be kept in mind when considering development.

1.6.1. Surface Water

ECA (2) (no date) reports that the possible effects of man-made lakes include change in an area's micro-climate, earthquakes due to loads on weak substrata, diseases, water weeds, silting up of lakes, and removal of fish from coastal areas.

Dams can increase local catches in their reservoirs. However, this may be only temporary. The catch in Lake Volta dropped one fifth in 1971 (Sterling 1972). The reasons may be overfishing, predatory fish overindulging, waterweeds cutting off air and light for fish life, etc.

Sterling (1972) describes in detail some of the water-borne diseases that may spread because of dam construction. The black fly simulium, the cause of river blindness, needs fast flowing water with a lot of oxygen. They inhabit the areas downstream of dams where the water comes out of the sluices. Calm lake surfaces and twisting shorelines are breeding grounds for malarial mosquitos, yellow fever flies, and snails which carry diseases.

Irrigation projects also sometimes do more harm than good. If improper drainage facilities are provided, soil salinity problems can result (UNESCO/FAO 1973). Irrigation projects may also be harmful because they may cause too large a population to settle and multiply. Eventually, the land may not be able to support them because of increased salinity, water logging or reservoir silting. However, it should be noted that this problem has not arisen at the few large-scale irrigation projects in the area. For example, the reason the French Niger Delta irrigation scheme was not very successful was because they could not get people to settle there.

Sterling (1972) also describes some of the harmful social effects that result when people are displaced because of dams. People find their entire way of life changed. They are often placed on inferior land. They become "unselfreliant, possibly aggrieved, and dispirited."

A major physical problem of man-made lakes is, of course, evaporation. In Mauritania, the loss from a reservoir would be ten feet per year.

Sterling (1972) points out another problem of water reservoir development. Oftentimes, high energy producing projects are built when there is no physical or economic structure to use the power such as grids, factories, workers, communication, transportation, markets, etc. To develop these necessities adds large costs to any project. As an alternative strategy, Sterling (1972) suggests building thermal power stations one at a time as consumption grows. (This is an interesting proposition and should be looked at.)

1.6.2. Groundwater

Ottaway (1973) reports that some well drilling projects in West Africa have resulted in lowering of the water table (which, by destroying surface vegetation, can cause erosion), and have attracted nomads whose herds destroy vegetation and expand the desert.

Groundwater use in coastal areas has also caused salt-water intrusion. This has happened in Dakar, Senegal. Another problem hampering groundwater development in West Africa is uncertain recharge.

A problem hampering water resource development in general in the area is not enough indigenous technical expertise and water organizations.

1.6.3. Experimental Work of Possible Value to Water Resource Development

Described in AID (1972) are many innovations that may be useful for water resource use and development in the area. These include:

1. Climate modification.
2. Various surface sealers (oil and rubber), artificial and natural windbreakers, and new forms of vegetation to control desertification.
3. Dew collection, using nylon strips hung a few inches above the soil.
4. Evaporation and evapotranspiration suppressants.
5. Irrigation by underground plastic pipes.

6. The use of asphalt barriers placed .6m below the surface to retard water percolation.
7. The use of "micro-catchments" which are small depressions in the soil so designed that runoff will reach plants growing in the center.

Others (Cook 1973, Adams 1973) have suggested using the ERTS satellite to conduct a resource survey; the use of solar energy, particularly solar pumps; and wind power. Desalination has also been suggested (Nouadhibou and Nouakchott, Mauritania, presently have desalination plants). The project staff has visited the experimental solar pumps at Niamey and Ouagadougou, and has detailed information available. Cluff and Dutt (1973) describe water harvesting schemes experimented with in Arizona that may be beneficial to the region.

Concerning these and other proposals suggested by an AID study, L. Lemoine, the former head of technical services of the Interafrican Committee for Hydraulic Studies, made the following comments:

"(a) The desalination of seawater and brackish water is the only hope many regions have of ultimately obtaining adequate water resources.

"While techniques in this regard have made substantial progress, the cost of obtaining such water remains particularly high. In addition to the now standard techniques of flash evaporation, freezing and the use of semi-permeable membranes, my feeling is that simpler technological processes which make use of solar energy should be studied. Such methods have already been used in various parts of the world, in particular, certain Mediterranean islands. Recourse is had to simple materials and construction procedures which can be applied in the developing countries.

"(b) Reducing losses due to evaporation from water surfaces is another important research subject for the countries of tropical Africa.

"The topography is usually highly unfavorable for building dammed reservoirs, and hydrological assessments show that from 50 to 90 percent of theoretical reservoir capacity is lost through evaporation. Even if such losses were reduced to a relatively slight extent, the amount of available resources could be expanded. Although a great deal of research on the subject has been undertaken throughout the world, so far no method capable of being used on an industrial scale has been discovered for dammed reservoirs.

"The body best qualified to co-ordinate and combine research programmes on this score is thought to be the International Commission on Irrigation and Drainage [1], which has published and regularly updates a world summary of studies on the subject.

"(c) The creation of subterranean reserves through underground nuclear explosions is certainly a promising technique deserving of prompt study, since it would enable large quantities of water to be stored and much of the above described loss by evaporation to be avoided. Such research would be of the greatest value for the African countries of the Niger basin previously referred to, where extensive regions are almost entirely lacking in groundwater resources and in facilities for storing surface water.

---

[1] International Commission on Irrigation and Drainage (ICID), 48 Nyay Marg, Chanakyapuri, New Delhi, India.

"The Interafrican Committee for Hydraulic Studies[2] with the help of the International Atomic Energy Agency and the French Atomic Energy Commission, has developed a programme of study and research in this field which we feel deserves to be considered by O. E. C. D. experts and by the various financial bodies which can contribute to its implementation.

"(d) Induced rainfall

"The authors of the report believe little can be expected from weather modification in the foreseeable future. We realize, of course, that the results of cloud-seeding experiments conducted in various countries have been the subject of considerable argument and have aroused the misgivings of American technicians in particular.

"Yet the stakes are so high for the countries of the Sahelian zone in West Africa, that we consider it imperative instead to stress research and experimentation in this field. As evidence, we submit the results obtained by Australian research staff, leading the Australian authorities to organize cloud-seeding campaigns on an operational scale.

"The object is not to modify substantially weather conditions over any very large region, but to develop methods of spot action, so that a given point in time, the slight extra amount of rainfall needed to save the crop of some specific area can be induced.

"Here again, an extensive research and experimentation programme has been set up by the Interafrican Committee for Hydraulic Studies with the help of the French Aid and Co-operation Fund. A first run of tests is to take place in 1974,

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[2] Interafrican Committee for Hydraulic Studies (Comite Interafrican d'Etudes Hydrauliques) (CIEH), P. O. Box 369, Ouagadougou, Upper Volta.

but it is already quite clear that operations should be carried over to ensuing years, since not all of the problems can be solved in a single campaign. The obstacles which must be overcome are enormous, and should the results of the first run be disappointing, the conclusion must not be that progress is unattainable, but rather that inadequate resources are being used.

"Only a few specific points have here been raised, and clearly many other fields call for similar research since we must agree with the author's statement of page 33 of the report that the reason why 'research has lagged in this field (is) because the advanced countries have not, generally speaking, suffered from insufficient water.'"

1.6.4. Some Recommendations of Others for the Region

There are several sets of recommendations concerning water resource development. Cook (1973) reports that a French hydrologist recommends that "varying flows of rivers and streams, at least in the upper river basins, must be regularized" to remove the Sahelian population from the precarious economic situation. ECA (no date) is also concerned about the fluctuation of water in those areas, as they state that water fluctuation is the biggest factor in the production system, social organizations, and relations. (It should be noted that this statement also implies that controlling the fluctuations may cause serious social changes that should be investigated.) Water fluctuations are traditionally controlled by dams, reservoirs, etc. However, Church (1970) and Sterling (1972) argue against large projects such as these as they often lead to serious problems (see previous section).

Another recommendation for the area is that the number of water points* for livestock be increased to prevent overgrazing of areas. Artificial ponds or hafirs (of 10,000 to 80,000 cubic meters capacity) to store rainwater three to four months as well as wells could be used. To prevent seepage from the hafirs, AID, OST (1972) recommends lining them with plastic. Evaporation loss is also a problem. Dekker (no date) suggests irrigating fodder for beef, since livestock raising is one of the natural vocations of the area and demand is high.

Dumont (1966) suggests using multi-purpose reservoirs for aquaculture. Aquaculture is practiced in Cameroon and the Congo, and was once practiced in the Sahel-Sudan Region.

Since surface water evaporates quickly in the area, the possibility of underground storage of water in aquifers or caverns should be investigated.

Cederstrom (1974) in a personal communication has recommended the following:

"Rather than focussing entirely on large perennial rivers and big dams, ... consider what might be done with important short lived streams that feed the large rivers or become lost in the desert.

"First, as used extensively and highly successfully in Aden, there are dams on ephemeral streams that spread out water on agricultural land. These dams can be designed to pass through the initial flow carrying boulders and trash, but, as the stream level rises, ports will allow water to flow onto nearby agricultural lands. At still higher flow the excess water will pass downstream. These are relatively "small" structures that would immediately bring

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*U.N. (1973) notes that such watering points may enable the nomads to settle at centers where their children may attend school, where they may receive medical attention, etc.

agricultural lands into production or radically upgrade some lands already in use.

"Further, there would be advantages in acting as flood control structures. Flood peaks on major rivers would be minimized, but in allowing minimum flow at all times, low flows would be little affected. It seems likely that siltation of major streams would be reduced, thus prolonging the life of conventional dams on major streams. Obviously, there would be some groundwater recharge that might be extremely helpful in some areas.

"No disadvantages of displacing people, mosquitos, etc. would accrue from construction of these "flood water irrigation dams".

"Another technique is spreader dikes. These are purposely leaky dams that slow down the flow of water in intermittent streams (flow perhaps only 1 or 3 times a year). In slowing the flow, there is greater opportunity for soil saturation making possible higher yields than previously, or perhaps making possible certain crops that could not be grown otherwise. These structures can be built of native rock carefully stacked, require no cement, and can be built by local labor. These structures were used to good effect by the Romans; new ones have been built in Libya by USAID and they are in use in some places in the USA.

"It is suggested that at least 10% of surface storage investment funds be allocated to such structures. This figure may be wildly conservative with respect to getting benefits widely dispersed and directly affecting the common man.

"The structures described would not fit in true barren desert or perhaps not in essentially humid areas. Where rainfall is infrequent but heavy, they would have great value."

U.N. (1973) recommends that the following statistical information be gathered on groundwater in Africa:

"1. Groundwater resources available for the water supply of towns and industrial areas that have no access to large surface-water resources. The groundwater resources, both potential and exploitable, should be compared with the estimated requirements;

2. Inventory of water requirements and available water resources in villages. Such an inventory is already well advanced in many countries, particularly in western Africa. In some cases precise maps have been prepared, as was done in Togo as part of the United Nations groundwater exploration project;

3. Location of areas where wells should be drilled to obtain water for livestock, with account taken both of requirements and of the need to protect certain pastureland; and

4. Inventory of areas in which irrigation using ground water is feasible and economically efficient.

5. In addition, a study of the cost of African groundwater for irrigation and its impact on the profitability of agricultural crops for internal consumption and for export should be undertaken without delay on the basis of data now available on agricultural production."

U.N. (1973b) recommends in their African regional plan, the following:

"1. Determination of water balance, all water resources, and their utilization.

2. Improvement of water storage conditions in suitable areas, elimination of waste, and reduction of natural losses.

3. Protection against water pollution.

4. Establishment of national bodies to administer water resources.

5. Cooperation between nations in developing water resources.

6. Application of modern methods to inland and coastal fisheries to aid in solution to the problem of protein deficiencies.

7. Water erosion control.
8. Study of the problems of irrigation, drainage, reclamation, etc.
9. Comparative study of irrigation methods.
10. Determination of water requirements of crops.
11. Improvement of urban and rural water supply.
12. Training of water technicians."

1.6.5. Some Effects of Present Drought

Rosenthal (1973) reports that the present drought is the worst in 60 years. There have been four years of subnormal rainfall. Some of the effects of the drought are listed below from Rosenthal (1973) and Cook (1973).

1. Great loss of livestock in area. The range is reported from 80% in Mali to 33% in Niger and Chad. Since 90% of the people in the Sahel depend upon livestock, this is disastrous. (Billings (1973) notes that these losses may be exaggerated.)

2. Crop losses are large. In Mauritania, sorghum production is down 75%. In Senegal, grain production is down 50%.

3. The Senegal River, during its last flooding (July-August 1972), failed to rise above the lower minor river bed. As a result, planting of crops covered scarcely one tenth of the normal area.

4. The fish catch in the St. Louis region has decreased from 30,000 tons to 15,000 tons.

5. So much water has evaporated from Lac du Guier, Dakar's water supply source, that the city is endangered.

6. The Niger River did not flood the area between the Bani River and the river in the region of Djenne.

7. At Niamey, Niger, it is possible to wade across the Niger River for the first time in memory.

8. A dry channel continuously leads from the Niger River to Tombouctou. The river is also so low that it is not navigable in this area. (It is usually navigable from January to April in this area.)

9. The water level in Lake Chad is so low it has fragmented into 4 separate sections. Millions of fish have been lost as a result.

10. Thousands of wells have dried up because of lower groundwater levels.

PART II: SYSTEMS FRAMEWORK FOR ANALYSIS OF WATER REQUIREMENTS AND SUPPLIES IN THE SAHEL-SUDAN REGION

2.1 Introduction to Part II

This part of the report contains the systems models developed to provide an analytic basis for the planning framework. This framework serves as a guide to the allocation of aid for the development and management of water resources in the region.

The basic approach of the project, it will be recalled, is to examine several broad alternative development patterns. The water resources systems work has been done to fit into this pattern. What follows summarizes the scheme which was developed and will serve as a guide to the contents of this part of the report.

A development pattern is a projection of the future that reflects particular types of change, seen as desirable, inevitable, or probable. To relate such overall projections to the water sector, we have adopted the following approach. For any projection of the future, we allocate the economic activity associated with that projection -- agriculture, industry, etc. -- to various subareas of the region, chosen as appropriate for water resources planning purposes. Then, water use coefficients are applied to the sectoral production projections to obtain total projected water use in every region. The water use is adjusted for reuse within each region. All of this is accomplished with a FORTRAN-coded projection model described in this part of the report.

This provides forecasts of water demand distributed by region, and by economic sector within each region. As the next step in the systems framework, the supply model is used to estimate broad water resources, investment and management strategies to meet the demands. The calculations are carried out by a mathematical programming model, described in this part of the report. The model is of a class known as "mixed-integer" programming models, and is coded for two seasons, wet and dry, and for various sources of supply.

The forecasting model and the supply model should be utilized in planning applications in an interactive way. The water demands associated with a desirable pattern of development are examined first using the supply model. If they are too unrealistic the supply model will indicate either that they cannot be met physically, or that the levels of expenditure required to do so are higher than are likely to be borne by the countries of the region, even with outside help. If this turns out to be the case, forecasts have to be modified in ways that bring them down to what appear to be reasonable levels of demand. While it is difficult to estimate exactly what levels of investment funds will be available for the water sector, we can infer something about this from the projections of aggregate investment in the region discussed elsewhere in the project report (See Annex 1: Economic Considerations). We know that many types of investment are required in the region; hence, we know that total investments in water resources cannot be more than some reasonable fraction of total investments forecast.

There is another form of interaction which is essential in application of the models described here. That is interaction between the systems researchers and decision-makers in the region. Such interaction is required for two reasons. First, decision-makers must contribute to the planning process because adequate expressions of preferences are essential for obtaining useful results from the models, particularly in the selection of desirable patterns of development. Second, all models are somewhat abstract, and thus do not capture important elements of a real situation. The knowledge of local experts is essential in helping the modellers to adjust the form and parameters of the systems models to take into account as best they can the important real aspects of the situation. Results obtained from models such as those developed here are really not entirely adequate for applications unless these types of interactions have taken place. (This is true for other planning methods also.)

The use of the models described above embodies a kind of loop in the systems framework of the overall project. First, there is a projection of a development alternative. Then, for the water sector, the demand and cost of supply implications of this alternative are examined and can be compared with the overall investment in the region as forecast by the economic model. Then, the investment requirements for all sectors (including water) that seem to be implied by the initial development projection can be aggregated, in principle at least, and compared with the initial assumptions of the projection to determine whether the initial development alternative appears to be a reasonable one. Hence, there needs to be interaction not only within the water sector models, but also in the analytic framework of the project as a whole.

The system of models described here can be used in a somewhat different way after exploration of forecasts and costs of supplies has been undertaken. The mathematical programming model is coded so that it can be used in net benefit maximizing form. Hence, after some general notion of reasonable future levels of demand has been obtained, benefit coefficients can be estimated for supplying water to various uses, and the model used to estimate the optimum net benefits in the system, and also whether, given the benefit and cost coefficients used, benefits exceed costs. The model in this form has another very important use, which is that constraints on the total amount of investment resources (indigenous and external) can be incorporated into the model to yield solutions giving the "best" allocation of limited investment resources in the water sector. In this form, the model should also be used iteratively with both local technical personnel and local and international (aid) decision-makers.

The work described in this part of the report is presented as follows: the methods used to divide the region into water planning areas are described in Section 2.2. Then, the structure of the FORTRAN forecasting model is described, and sample runs of the model are discussed in Section 2.3. Next, the supply model is described in Section 2.4. The results of illustrative applications of the supply model are presented in Section 2.5.

2.2 Delineation of Water Resources Planning Areas*

One of the most important tasks in large-scale water resources planning is the delineation of suitable water resources planning areas. On the one hand, areas for planning should not be so large that the aggregation involved distorts planning conclusions. On the other hand, if too many areas are used, the costs of planning become high and, in particular, computational costs become high. The approach used in the project to select water planning areas was to examine the region from the standpoint of a list of criteria developed for the purpose, to draw overlay maps based on the various criteria, and then, by overlaying the maps, to estimate the boundaries of suitable water resources planning areas based on the extent to which the various criteria used indicate generally similar boundaries. Compromises between criteria had to be made, of course, but with this approach it was possible to delineate suitable areas.

The regions developed using our initial criteria for selection were modified as the details of the supply model were developed. In this section the initial list of criteria is shown. Then, the nature of the modification to take into account the details of the supply model is described and a map of planning regions currently used for Mauritania is provided. (Figure 2.5-2)

*This section is based in large part on work by Ernest Brown and Kenneth Strzepek.

The criteria that were used to establish the initial set of planning regions are listed here. These criteria include physical, climatic, economic, social, and political factors.

Criteria for Selection of Initial Planning Regions

1. Perennial/Seasonal Flow Characteristics
2. Basin Boundaries
3. Economic Commonality (e.g., similar cost of supply functions)
4. Aquifer Boundaries
5. Recharge/Fossil Groundwater
6. Conjunctive Sources of Groundwater and Surface Water
7. Decision-Making Boundaries (including political boundaries)
8. Data Quality Commonalities
9. Commonalities between demand and supply areas, and between particular uses (e.g., navigation) and supply areas
10. Water Quality Commonalities
11. Health Problem Boundaries
12. Rainfall Regions

The transparent overlay maps were developed for these criteria and were aligned by means of cross marks on the originals and also by country boundaries where these were included on the maps. Country names, river names, etc. showed through from a base map on which the overlays are used.

When the initial supply subareas were used with the supply model (discussed later on) in a sample formulation using Mauritania, it was found that it was desirable to increase emphasis on certain criteria, including especially physical criteria. In addition, supply and demand distribution were used to help delineate final areas.

The supply and demand distribution is important to consider because the water supply mocel assumes that the cost of supplying every user in a subarea from a particular source in that subarea is the same. Therefore, the situation of having a source at the border of a subarea should be avoided when possible. An example of such a situation would be including Nouakchott in the same subarea as the Senegal River. Obviously, demands close to the river (such as irrigation) can be supplied considerably less expensively than demands far from the river (such as Nouakchott). The use of an average cost would thus not be entirely representative. (However, there are subareas where this situation arises, and in these, the cost of supply used is the cost of supply to the center of the subarea or "center of mass" of the demand. Sensitivity analysis of the cost is used to further analyze the solution. This is explained in greater detail in the Supply Model discussion (Section 2.4-3).)

The supply and demand distribution is also important in analyzing local effects at the water sources. An example of such a local effect is the drawdown that occurs at an aquifer because of the slow response time of the aquifer compared to a surface water source. The water supply model does not explicitly consider such local effects and assumes that sources are used such that local effects are generally equalized throughout. Such a situation would occur only if the users are evenly distributed throughout a subarea. Therefore, attempts were made to choose subareas with equally distributed demands, and, if necessary, to model the uneven distribution and unequal localized effects by placing higher costs on the use of some sources above certain amounts.

2.3 Requirements Forecasting

2.3.1 Introduction: An essential element in water resources planning is the estimation of water requirements for future years.

There exist sophisticated methods of projecting water requirements, as for example the input-output based NAR demand model (Schaake and Major 1972). However, since the Sahel-Sudan countries have no regional input-output tables and since data series are not long enough to justify the application of elaborate regression techniques, it seemed best to develop requirements estimates by simple techniques of applying selected water use coefficients to projections of basic economic and demographic variables. Projections of water requirements were made by such techniques for 1980, 1990, and 2000, and in addition, estimates of current use were made where data are not available. (Estimates of current use provide a basis for current demand/supply comparisons and also demonstrate the implications of the water use coefficients used for the projections to future years).

The success of this approach depends on the suitability of the computational format used, and on the input data [illegible] go into the runs that are made. As explained in the introduction to this part of the report, this model and the supply model should be used iteratively in planning applications to insure as best as is possible that the data used, and the manipulations performed on the data, reflect the planning problems of the region.

In this section of the report, the nature of the demand or requirements forecasting model is described. (A users' manual for the program and listing is provided in Appendix A.) Two sample runs are described, and samples of input and output are provided. Only a sample of data sources and parameter estimates is listed here, of course, since in future uses of the model, input data will (properly) be utilized from the best available sources, which will change over time, and hence the estimates of model parameters will also change.

2.3.2 Description of the requirements forecasting or demand model: The model is a general model that can be used to forecast requirements for any resource, for any number of countries and regions, and for any number of forecast years. The computer program that constitutes the model performs simple arithmetic operations upon input data to yield "requirements" or "demand" forecasts for a given use in several subareas for forecast years. The use of the model for the water sector of the Sahel-Sudan region is described here.

Most input data for the region are available by countries, and the model is written to accept country data. Forecasts of activity in economic and demographic sectors by country are the basic input data for the model. These forecasts can be entered in two ways: either as numerical forecasts for each forecast year, or as an initial value plus a growth rate. This feature of the model is provided to assist in communication with decision-makers who might wish to think in terms of growth rates rather than in terms of absolute values.

These forecasts of activity are then allocated by the model to water resources planning regions within each country. This is accomplished by multiplying the forecast value (say industrial production by weight of product) by a vector of allocation percentages which allocates 100 percent of the production to designated areas. The allocators can be different, in the model, for each economic sector (user) and for each forecast year. After this has been done, water use coefficients are applied to the production figures for each sector for each subarea. These coefficients are defined in terms appropriate to the activity: for population, liters/person; for industrial production, liters/ton, etc. The water use coefficients can vary over time, by sector and by area. This operation yields gross water demands by sector for each subarea in each forecast year. These projections, appropriately summarized, constitute the basic input to the supply model, with the exception that the gross demands in each sector must be reduced to account for reuse of the water within the region -- e.g., in the case of factories located along the same stream.

The information required to run the model is summarized as follows:

Varibles

Number of countries

Number of regions being studied

Number of water-consuming sectors (or users)

Number of time segments (four in this study: 1970, 1980, 1990, 2000)

Length of time periods (in this study, 10 years)

The number of water-consuming sectors (in this study, the sectors are:

- urban population
- rural population
- industrial production per year in tons by subsectors (13 sectors used in initial runs)
- electrical energy production in kwh per year
- irrigated agricultural land in hectares
- animal population by type (6 types used in initial runs or as animal unit aggregates)

Data and Basic Projections

Basic economic and demographic data are read in for each country by water-consuming sector (24 sectors used in initial runs) for the first time segment (1970) and as projected data for the future time periods.

Regional Allocators. Regional Allocators are read in for each country by sector, time, and region. These coefficients distribute sector of activity among areas.

Water Use Coefficients. These are read in for each country in the same way as the allocators: by sector, time, and region. The water-use coefficients represent the water need of each water consuming sector per unit of quantity (e.g., 16,000 l/ton for the fish industry).

Reuse coefficients. These are read in for each country in the same way as allocators and water-use coefficients are (by sector, time, and region). The reuse coefficient represents the proportion of the water use that is reused within the same region for each sector.

All of these variables (allocator, water use coefficient and reuse coefficient) can be expected to change over time; but in initial runs we assume them to remain constant for the sake of simplicity.

It can be seen that a large number of input numbers are required to operate the model. An important advantage of the model is that it permits the rapid examination of the effects on water requirements in the region of changing any single input number or any group of input numbers; hence, in iterative use of the model, the implications of suggestions from technical personnel and from decision-makers for water use in the region can be easily and quickly explored.

2.3.3 Development of the example runs of the model

Several runs of the demand projection model have been made in the course of the project. In this section, the development of data for two related sample runs is discussed, and the results of one of the runs, which forms the basis of the case study presented in Section 2.5, are displayed in 2.3.4. Because the demand projection model is written in a general format, there is no one set of data sources that is necessarily appropriate for use in the model.

It is designed to use data inputs from a variety of sources, including estimates by technical personnel and political decision-makers. In applying this model to an actual planning operation, it would be expected that many runs would be made.

The data sources described below were used to make two runs of the demand model, with the same coefficients and projections generally, except that one run of the model (called Run 1, in the description that follows) emphasized rural development, and the other (Run 2) placed relatively heavy emphasis on urban development. The results of a version of the former run (several versions of a particular base run are typically made with models such as the one under discussion) are presented in the next section, and these results form the basis of the case study of Section 2.5.

Data Sources for Sectoral Forecasts

Demographic: The population figures used in the runs were taken from the Interim Report, Part II, p. G51. These figures are for total population for each country to the year 2000. For the purpose of utilizing the figures in the two runs described here, it was necessary to estimate urban/rural population figures. The percentage allocators used for this purpose are shown in Tables 2.3-1 and 2.3-2.

Industry: Fifteen primary industrial sectors were utilized in the model runs: meat, sugar, fish, rice, milk, beer, wheat, leather, textiles, cement, brick, iron, copper, energy, fertilizers. While this is a fairly fine breakdown for the industrial sector, given the current economic structure of the region, nonetheless the breakdown is useful because it allows for specific comment on the results from technical personnel and decision-makers; and, in addition, certain industries are of particular importance, or will be in the future, to certain water using areas, and therefore should be broken out, even

TABLE 2.3-1

Percentages of Urban-Rural Population for the Sahel-Sudan Region

RUN 1

	1970	1980	1990	2000
CHAD				
Urban	9	14	18	20
Rural	91	86	82	80
MALI				
Urban	9	14	18	20
Rural	91	86	82	80
MAURITANIA				
Urban	12	20	26	30
Rural	88	80	74	70
NIGER				
Urban	5	11	15	18
Rural	95	89	85	82
SENEGAL				
Urban	30	35	38	40
Rural	70	65	62	60
UPPER VOLTA				
Urban	8	14	20	25
Rural	92	86	80	75

Source: --Interim Report, Page 67

TABLE 2.3-2

Percentages of Urban-Rural Population
for the Sahel-Sudan Region

RUN 2

	1970	1980	1990	2000
CHAD				
Urban	9	16	24	30
Rural	91	89	76	70
MALI				
Urban	9	18	26	30
Rural	88	78	68	60
MAURITANIA				
Urban	12	22	32	40
Rural	88	78	68	60
NIGER				
Urban	5	14	23	28
Rural	95	86	77	72
SENEGAL				
Urban	30	38	46	50
Rural	70	62	54	50
UPPER VOLTA				
Urban	8	18	28	35
Rural	42	82	72	65

Source: Staff Estimate

though they might not be important in a country taken as a whole.

The production figures for these industries were taken from EDIAFRIC (1972), and converted into tons, kiloliters, or kwh as appropriate for the model. Figures for electric power generation represent production, not capacity; estimated growth rates for energy production are based on information relating to plans for construction of new power plants between 1970 and 1975 (EDIAFRIC (1972).

Agricultural: For these runs, which relate to the agricultural development alternative studied in the project, fairly substantial amounts of irrigated land were projected. The figures were derived from the material presented in Part I of this report. The unit amount of water per hectare required for irrigation in the future is assumed to be equal to current figures. Obviously this is a figure that can be assumed to change over time.

Animal: Six types of animals are explicitly counted as water users in the region: cattle, sheep, goats, horses, pigs, donkeys, and camels. The source of animal population data was the FAO Production Yearbooks series, which provide data converted into "animal units"(amu).

Sources of Estimates of Regional Allocators, Water Use Coefficients, and Correction (Reuse) Factors

Regional allocators of population were derived by taking the urban (or rural) population in each administrative area and redistributing this population to water planning areas on a judgemental basis. (Source of the figures of population: French Ministry of Cooperation (1970). Then, the proportion of each population type in each water planning area was compared to the total of that population type (urban or rural) in each country, and the proportional allocators calculated. The sum of allocators for each population type within each country is 1, i.e., all population of each type in each country is allocated to a water planning area. These allocators, as with many of the other data inputs, could easily be assumed to shift over time, although such shifts were not incorporated into the two runs under discussion.

The water use coefficient for personal consumption was taken to be just under 1/2, on a judgemental basis, of the recommended per capita use of 16 gallons per day per person (= 72.7 liters) in the "Water Resource Survey and Development Project of Central Senegal", AID (1969). Good information on domestic use is not available for the region; as a result, in detailed applications of the model this coefficient should be subjected to sensitivity analysis.

The correction factor for human use in each area is assumed to be 1, i.e., there is assumed to be no reuse of domestic water.

Industries and Electric Power Generation: Regional allocators for industry and electric power generation were developed in the same way as for human population, with the basic production data coming from EDIAFRIC (1972). The allocators for many industrial sectors for some regions are, of course, zero, since no industries in that sector are located in those water planning areas. The industrial regional allocators are assumed constant over time.

The water use coefficients were based on figures in Todd (1970). Figures were available in the case of most industrial sectors for Africa. When African figures were not available, figures from other developing countries were utilized. The coefficients are in liters/ton for industries except for power (liters/kwh) and milk and beer processing (liters/kiloliter).

Correction factors in the case of the industrial sectors might well be other than 1 (i.e., gross estimated use would be reduced by the correction factor), but for simplicity in the runs shown, these factors were taken as 1.

Agriculture: The regional allocators for agriculture were derived from knowledge of the present locations of irrigated areas in the entire region, and the allocators were assumed to be unchanged over the forecast period.

The water use coefficient used per "animal unit" was taken from a staff estimate to be 66.7 l/day/amu.

The correction factors were assumed to be 1.

The water use coefficients for the users in Mauritania and Senegal are summarized in Table 2.3-3.

Industrial, Power, and Agricultural Sector Growth Rates for the Two Runs: Two potential growth rates for the industrial, agricultural, and power sectors were estimated. The first growth rate was derived from the rates given in EDIAFRK (1972) or other sources for that sector in the decade from 1960 to 1970. The second growth rate was taken to be 1/3 of this rate. Then, sectors were divided into two groups: those whose growth would probably be linked more closely with urban areas than with rural areas, and industries for which the reverse would probably be true. (The allocations are, of course, judgemental, since many if not most sectors are directly or indirectly dependent upon both urban and rural growth). For Run 1, the rural emphasis alternative, the sectors thought to be more closely associated with ruralization were assigned the higher growth rates estimated, and the sectors associated with urbanization were given the lower growth rates calculated. For run 2, the run emphasizing urban development, the reverse was true.

The sectors (with sector numbers as used in the program) assumed to be linked more closely with rural areas are: meat(3); sugar(4); rice(6); milk(7); beer(8); leather(10); fertilizer(17); irrigated land(18); animal husbandry(19-24).

Table 2.3-3

Water Use Coefficients, Senegal and Mauritania

			Water Use
Human	1	Urban Population	30.0 l/day/capita
	2	Rural	30.0 l/day/capita
Industrial	3	Meat	200 l/ton/year
	4	Sugar	1800 l/ton/year
	5	Fish	16000 l/ton/year
	6	Rice	
	7	Milk	
	8	Beer	
	9	Wheat	
	10	Leather	
	11	Textile	
	12	Cement	550 l/ton/year
	13	Brick	
	14	Ore-iron	4200 l/ton/year
	15	Copper	3100 l/ton/year
	16	Energy	5 l/KWH
	17	Fertilizer	270,000 l/ton/year
Crops	18	Crop Land	1 l/ha./second
livestock	19	Cattle	66.7 l/amu/day
	20	Sheep and Goat	66.7 l/amu/day
	21	Horses	66.7 l/amu/day
	22	Pigs	66.7 l/amu/day
	23	Donkeys	66.7 l/amu/day
	24	Camels	66.7 l/amu/day

The sectors (with sector numbers as used in the program) assumed to be more closely linked with urban areas are: fish (5); textile (11); cement (12); brick (13); iron-ore (14); copper (15); electric energy generation (16).

2.3.4 Sample of demand projection model output: This section describes the model output showing part of the printed output for Senegal for the year 1990 when ruralization is emphasized.

Table 2.3-4 shows the growth rates or estimated production by sectors for country 1 (Senegal) for the years 1970, 1980, 1990, and 2000. The sectors are those listed in Table 2.3-3. If the number listed is greater than 10.0, it is a quantity; otherwise it is a growth rate. The unit of the quantities corresponds to those in Table 2.3-3 (i.e., population in capita, meat in tons/year, etc.).

Table 2.3-5 shows the estimated future quantities in the various sectors for the quantities for which growth rates are specified in Table 2.3-4. For example, in the year 1990, it is projected that 32,974 tons/year of meat (Sector 3) will be processed in Senegal (Country 1).

Table 2.3-6 shows the Regional Allocators (RALL), Water Use Coefficients (WCOEF) and Correction (Reuse) Factors (REGDF). The regions refer to the subareas of Senegal. These are for revised area boundaries rather than for the boundaries shown in Figure 2.5-1.

Shown in Table 2.3-7 is the actual projected quantity of water use by country, region (or subarea), sector (or user) and time period. Shown is the output for Country 1 (Senegal), regions 1 and 2, for time period 3 (1990). The column on the far right indicates the final water demand value after reuse. (These figures are equal to the "water need" figures because in these runs the reuse factors in all sectors was taken to be 1.)

2.4 Supply Model

2.4.1 Introduction: The objective of the model is to help

TABLE 2.3-4

COUNTRY	SECTOR	GROWTH RATE OR ESTIMATED QUANTITIES 1970	1980	1990	2000
1	1	0.1127E+07	0.1642E+07	0.2226E+07	0.2928E+07
1	2	0.2629E+07	0.3050E+07	0.3632E+07	0.4392E+07
1	3	0.2000E+05	0.2500E+00	0.2500E+00	0.2500E+00
1	4	0.2700E+05	0.4000E+00	0.4000E+00	0.4000E+00
1	5	0.7100E+05	0.1500E+00	0.1500E+00	0.1500E+00
1	6	0.0	0.0	0.0	0.0
1	7	0.0	0.0	0.0	0.0
1	8	0.0	0.0	0.0	0.0
1	9	0.0	0.0	0.0	0.0
1	10	0.0	0.0	0.0	0.0
1	11	0.0	0.0	0.0	0.0
1	12	0.2500E+06	0.1400E+00	0.1400E+00	0.1400E+00
1	13	0.0	0.0	0.0	0.0
1	14	0.0	0.0	0.0	0.0
1	15	0.0	0.0	0.0	0.0
1	16	0.3500E+09	0.1700E+00	0.1700E+00	0.1700E+00
1	17	0.1600E+07	0.4500E+00	0.4500E+00	0.4500E+00
1	18	0.8000E+04	0.6000E+05	0.1200E+06	0.2500E+06
1	19	0.1820E+07	0.5000E+00	0.5000E+00	0.5000E+00
1	20	0.5560E+06	0.4000E+00	0.4000E+00	0.4000E+00
1	21	0.1710E+06	0.1300E+01	0.1300E+01	0.1300E+01
1	22	0.1750E+05	0.4000E+00	0.4000E+00	0.4000E+00
1	23	0.7200E+05	0.4000E+00	0.4000E+00	0.4000E+00
1	24	0.3600E+05	0.4500E+00	0.4500E+00	0.4500E+00
2	1	0.1224E+06	0.2852E+06	0.4519E+06	0.6357E+06
2	2	0.8976E+06	0.1141E+07	0.1738E+07	0.2119E+07
2	3	0.0	0.0	0.0	0.0
2	4	0.0	0.0	0.0	0.0
2	5	0.3600E+05	0.5000E+00	0.5000E+00	0.5000E+00
2	6	0.0	0.0	0.0	0.0
2	7	0.0	0.0	0.0	0.0
2	8	0.0	0.0	0.0	0.0
2	9	0.0	0.0	0.0	0.0
2	10	0.0	0.0	0.0	0.0
2	11	0.0	0.0	0.0	0.0
2	12	0.0	0.0	0.0	0.0
2	13	0.0	0.0	0.0	0.0
2	14	0.8500E+07	0.2000E+00	0.2000E+00	0.2000E+00
2	15	0.8000E+04	0.2000E+00	0.2000E+00	0.2000E+00
2	16	0.1760E+05	0.2000E-01	0.2000E-01	0.2000E-01
2	17	0.0	0.0	0.0	0.0
2	18	0.0	0.5000E+05	0.1250E+06	0.2500E+06
2	19	0.1792E+07	0.5000E-01	0.5000E-01	0.5000E-01
2	20	0.1370E+07	0.1700E+00	0.1700E+00	0.1700E+00
2	21	0.2100E+05	0.1000E-01	0.1000E-01	0.1000E-01
2	22	0.0	0.0	0.0	0.0
2	23	0.9200E+05	0.2000E-01	0.2000E-01	0.2000E-01
2	24	0.8280E+06	0.2600E+00	0.2600E+00	0.2600E+00
3	1	0.4520E+06	0.8576E+06	0.1343E+07	0.1819E+07
3	2	0.4570E+07	0.5268E+07	0.6119E+07	0.7277E+07

TABLE 2.3-5

ESTIMATED QUANTITIES FOR VARIOUS TIME SEGMENTS

COUNTRY	SECTOR	VALUE	TIME
1	3	25680.500	1980
1	3	32974.406	1990
1	3	42339.965	2000
1	4	40279.277	1980
1	4	60089.637	1990
1	4	89643.187	2000
1	5	82490.187	1980
1	5	95839.875	1990
1	5	111350.000	2000
1	6	0.0	1980
1	6	0.0	1990
1	6	0.0	2000
1	7	0.0	1980
1	7	0.0	1990
1	7	0.0	2000
1	8	0.0	1980
1	8	0.0	1990
1	8	0.0	2000
1	9	0.0	1980
1	9	0.0	1990
1	9	0.0	2000
1	10	0.0	1980
1	10	0.0	1990
1	10	0.0	2000
1	11	0.0	1980
1	11	0.0	1990
1	11	0.0	2000
1	12	287568.312	1980
1	12	330782.125	1990
1	12	380489.812	2000
1	13	0.0	1980
1	13	0.0	1990
1	13	0.0	2000
1	14	0.0	1980
1	14	0.0	1990
1	14	0.0	2000
1	15	0.0	1980
1	15	0.0	1990
1	15	0.0	2000

TABLE 2.3-6

Coefficients for Demand Projections

COUNTRY	SECTOR	TIME SEGMENT	REGION 1	REGION 2	REGION 3	REGION 4	REGION 5	REGION 6	REGION 7	REGION 8	REGION 9	REGION 10	DATA TYPE
1	1	1	0.120	0.130	0.004	0.014	0.700	0.030	0.0	0.0	0.0	0.0	RALL
1	1	1	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	1	1	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	1	2	0.120	0.130	0.004	0.014	0.700	0.030	0.0	0.0	0.0	0.0	RALL
1	1	2	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	1	2	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	1	3	0.120	0.130	0.004	0.014	0.700	0.030	0.0	0.0	0.0	0.0	RALL
1	1	3	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	1	3	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	1	4	0.120	0.130	0.004	0.014	0.700	0.030	0.0	0.0	0.0	0.0	RALL
1	1	4	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	1	4	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	2	1	0.330	0.320	0.020	0.040	0.170	0.120	0.0	0.0	0.0	0.0	RALL
1	2	1	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	2	1	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	2	2	0.330	0.320	0.020	0.040	0.170	0.120	0.0	0.0	0.0	0.0	RALL
1	2	2	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	2	2	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	2	3	0.330	0.320	0.020	0.040	0.170	0.120	0.0	0.0	0.0	0.0	RALL
1	2	3	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	2	3	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	2	4	0.330	0.320	0.020	0.040	0.170	0.120	0.0	0.0	0.0	0.0	RALL
1	2	4	30.000	30.000	30.000	30.000	30.000	30.000	0.0	0.0	0.0	0.0	WCOEF
1	2	4	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	3	1	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	3	1	200.000	200.000	200.000	200.000	200.000	200.000	0.0	0.0	0.0	0.0	WCOEF
1	3	1	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	3	2	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	3	2	200.000	200.000	200.000	200.000	200.000	200.000	0.0	0.0	0.0	0.0	WCOEF
1	3	2	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	3	3	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	3	3	200.000	200.000	200.000	200.000	200.000	200.000	0.0	0.0	0.0	0.0	WCOEF
1	3	3	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	3	4	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	3	4	200.000	200.000	200.000	200.000	200.000	200.000	0.0	0.0	0.0	0.0	WCOEF
1	3	4	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	4	1	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	4	1	1800.000	18.000	1800.000	1800.000	1800.000	1800.000	0.0	0.0	0.0	0.0	WCOEF
1	4	1	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	4	2	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	4	2	1800.000	18.000	1800.000	1800.000	1800.000	1800.000	0.0	0.0	0.0	0.0	WCOEF
1	4	2	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	4	3	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	4	3	1800.000	18.000	1800.000	1800.000	1800.000	1800.000	0.0	0.0	0.0	0.0	WCOEF
1	4	3	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	4	4	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	4	4	1800.000	18.000	1800.000	1800.000	1800.000	1800.000	0.0	0.0	0.0	0.0	WCOEF
1	4	4	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF
1	5	1	0.0	1.000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	RALL
1	5	1	16000.000	16000.000	16000.000	16000.000	16000.000	16000.000	0.0	0.0	0.0	0.0	WCOEF
1	5	1	1.000	1.000	1.000	1.000	1.000	1.000	0.0	0.0	0.0	0.0	REGDF

TABLE 2.3-7

Demand Quantity by Sectors

Country	Region	Section	Time	Water Need		Corrected Water Need
1	1	1	3	8013742.00	LITER/DAY	8013742.00
1	1	2	3	35956368.0	LITER/DAY	35956368.0
1	1	3	3	0.0	LITER/YEAR	0.0
1	1	4	3	0.0	LITER/YEAR	0.0
1	1	5	3	0.0	LITER/YEAR	0.0
1	1	6	3	0.0	LITER/YEAR	0.0
1	1	7	3	0.0	LITER/YEAR	0.0
1	1	8	3	0.0	LITER/YEAR	0.0
1	1	9	3	0.0	LITER/YEAR	0.0
1	1	10	3	0.0	LITER/YEAR	0.0
1	1	11	3	0.0	LITER/YEAR	0.0
1	1	12	3	0.0	LITER/YEAR	0.0
1	1	13	3	0.0	LITER/YEAR	0.0
1	1	14	3	0.0	LITER/YEAR	0.0
1	1	15	3	0.0	LITER/YEAR	0.0
1	1	16	3	73759712.0	LITER/YEAR	73759712.0
1	1	17	3	0.0	LITER/YEAR	0.0
1	1	18	3	10264318000.	LITER/DAY	10264318000.
1	1	19	3	164991568.	LITER/DAY	164991568.
1	1	20	3	41267312.0	LITER/DAY	41267312.0

COUNTRY	REGION	SECTOR	TIME	WATER NEED (LITER)		CORRECTED WATER NEED (LITER)
1	1	21	3	76781632.0	LITER/DAY	76781632.0
1	1	22	3	1298881.00	LITER/DAY	1298881.00
1	1	23	3	5343966.00	LITER/DAY	5343966.00
1	1	24	3	2953000.00	LITER/DAY	2953000.00
1	2	1	3	8681555.00	LITER/DAY	8681555.00
1	2	2	3	34866800.0	LITER/DAY	34866800.0
1	2	3	3	6594881.00	LITER/YEAR	6594881.00
1	2	4	3	1081613.00	LITER/YEAR	1081613.00
1	2	5	3	1533437950.	LITER/YEAR	1533437950.
1	2	6	3	0.0	LITER/YEAR	0.0
1	2	7	3	0.0	LITER/YEAR	0.0
1	2	8	3	0.0	LITER/YEAR	0.0
1	2	9	3	0.0	LITER/YEAR	0.0
1	2	10	3	0.0	LITER/YEAR	0.0
1	2	11	3	0.0	LITER/YEAR	0.0
1	2	12	3	181930160.	LITER/YEAR	181930160.
1	2	13	3	0.0	LITER/YEAR	0.0
1	2	14	3	0.0	LITER/YEAR	0.0
1	2	15	3	0.0	LITER/YEAR	0.0
1	2	16	3	2261963520.	LITER/YEAR	2261963520.
1	2	17	3	0.0	LITER/YEAR	0.0
1	2	18	3	103680000.	LITER/DAY	103680000.
1	2	19	3	9899492.00	LITER/DAY	9899492.00
1	2	20	3	2476039.00	LITER/DAY	2476039.00
1	2	21	3	4606897.00	LITER/DAY	4606897.00
1	2	22	3	77932.87	LITER/DAY	77932.87
1	2	23	3	320638.06	LITER/DAY	320638.06
1	2	24	3	177180.00	LITER/DAY	177180.00
1	3	1	3	267124.75	LITER/DAY	267124.75
1	3	2	3	2179175.00	LITER/DAY	2179175.00
1	3	3	3	0.0	LITER/YEAR	0.0
1	3	4	3	0.0	LITER/YEAR	0.0
1	3	5	3	0.0	LITER/YEAR	0.0
1	3	6	3	0.0	LITER/YEAR	0.0
1	3	7	3	0.0	LITER/YEAR	0.0
1	3	8	3	0.0	LITER/YEAR	0.0
1	3	9	3	0.0	LITER/YEAR	0.0
1	3	10	3	0.0	LITER/YEAR	0.0
1	3	11	3	0.0	LITER/YEAR	0.0
1	3	12	3	0.0	LITER/YEAR	0.0
1	3	13	3	0.0	LITER/YEAR	0.0
1	3	14	3	0.0	LITER/YEAR	0.0
1	3	15	3	0.0	LITER/YEAR	0.0
1	3	16	3	0.0	LITER/YEAR	0.0
1	3	17	3	0.0	LITER/YEAR	0.0
1	3	18	3	0.0	LITER/DAY	0.0
1	3	19	3	49497456.0	LITER/DAY	49497456.0
1	3	20	3	12380190.0	LITER/DAY	12380190.0
1	3	21	3	23034480.0	LITER/DAY	23034480.0
1	3	22	3	389664.19	LITER/DAY	389664.19

decision-makers choose among water resources investment strategies in the Sahel-Sudan. The model is capable of doing this in the following ways:

1. Selecting the water supply sources that meet projected demands with the least cost.

2. Selecting the water supply sources and water uses that maximize net benefits, subject to constraints on total financing.

3. Providing a framework for the analysis of water resources issues.

The model is based upon a variation of linear programming called mixed integer programming. The model is of sufficient generality that it can be used to study the entire region at once; trade-offs can be made between investments in different locations and countries. However, it can also be applied to only one country or to some other subregion of the entire region. Each nation is divided into 3-6 subareas based upon the criteria discussed previously. The water sources available in each subarea are determined. These include river storage potential, yields of different aquifers, runoff, etc. The costs of utilizing each source are also estimated. The use of new technologies is modelled by either lower costs and/or increased yields. The demands are then inserted (determined outside the model if least cost, determined by model if net benefit maximation) and the model is operated. In cases where data are non-existent or not available, estimates are used. In this manner, broad water resource strategies are indicated.

The model provides a framework for analysis in several ways. The most obvious is that it forces the user to think explicitly about the problem definition, about solution alternatives, and required data. (This is common to all thorough systems analysis methods). The model can be used, for example, to identify areas where better data is needed, and to give some indication of the potential usefulness of this data based upon the sensitivity of solutions to it. In this manner priorities can be set on data collection. For example, it may be shown that groundwater development in a subarea will be too costly even if the best groundwater conditions exist.

The model also serves as a guide in deciding in which subregions more detailed water resource planning models should eventually be applied. The output of the regional supply model indicates broad water resources investment strategies. For example, for a given range of future demands, it might be predicted that a certain level of storage in the headwaters of the Niger River should be provided. The next step in planning might be the construction of a more detailed mathematical programming model that would be applied to the basin. This model would be used to select dam sites, irrigation projects, etc. The next planning step would probably be the use of a detailed simulation model of the basin to provide more detailed information on project dimensions and effects. Lastly, a budgeting and scheduling model might be used to study the implementation timing of the projects in the basin.

2.4.2 Water Supply Sources Modelled: The following are the water supply sources modelled:

1. Desalination of ocean water.

2. Precipitation. Precipitation can be either used directly (for example in rain-fed agriculture), recharged to aquifers, or else stored for later use. The storage can be either large-scale or small-scale. Large scale storage involves the catching of precipitation in unimproved or specially treated catchments and the storage of the runoff behind small dams, in farm ponds, etc. Small-scale storage is that done essentially by individuals using roof tops, cisterns, etc.

The water is caught in the same container it is stored in.

3. Importation of water from another subarea.

4. Withdrawals of surface water. (Surface water includes both rivers and lakes. The rivers may have reservoirs built on them).

5. Recycled water.

6. Groundwater. Recharge is also modelled.

2.4.3 Model Formulation: In the model the year is divided into two seasons; water flows and requirements are modelled within each season. The model, a mixed integer programming model, can be used in either a minimum cost or a net benefit maximization format.

The general schematic of a subarea is shown in Figure 2.4-1. (The general schematic shows that there must be at least enough water supplied in each subarea to meet total demands.) The surface water inflows to the subarea are upstream inflows, tributary inflows, and overland flow from precipitation. The "reservoir" controls that portion of the subarea above the reservoir; it is assumed not to control the drainage in the upstream subarea.* The size of the drainage area is controlled by the choice of the subarea location and area. (The "reservoir" may actually be several reservoirs controlling different drainage areas.) The outflow of the subarea equals the inflow to it minus the surface water withdrawal plus the return flow from users plus (or minus) base flow (or effluent flow). The value of the flow in the center or critical part of the subarea is also determined. This flow value may be subject to a low flow constraint for navigation, pollution control, or other reasons. In the schematic, imported water from another subarea is not assumed to enter surface water flows.

*DeLucia(1974) points out that this requirement is not necessary. Reservoirs controlling drainage in an upstream subarea or acting in series with upstream reservoirs could be modelled by using 0-1 integer constraints.

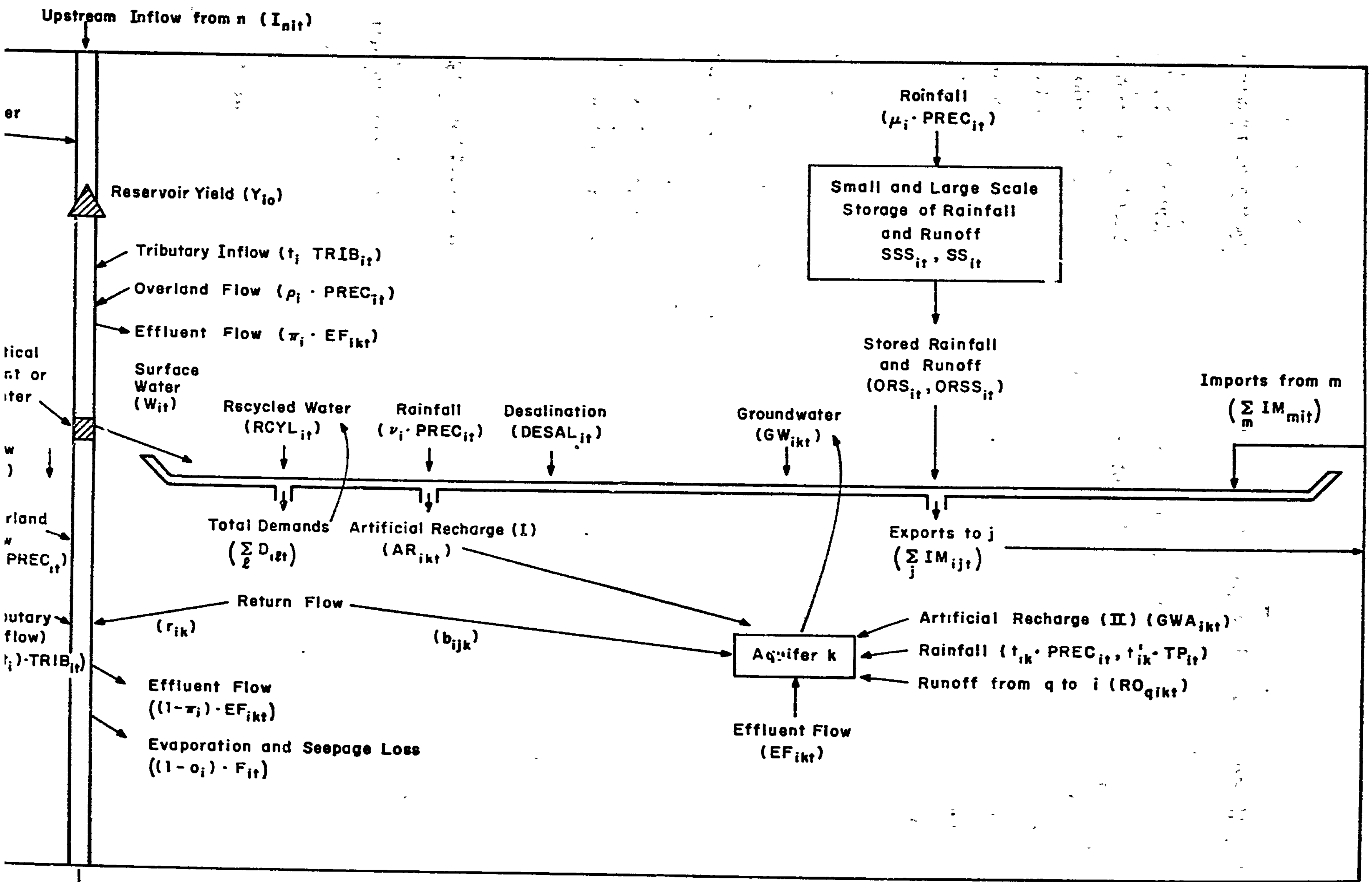

FIGURE 2.4-1 Typical Subarea

(However, if this were the case, the model can be adapted to account for it.)

The model considers two values of seasonal rainfall. The first (TP_{it}) is the total seasonal rainfall. The second ($PREC_{it}$) is the sum of the monthly rainfall of those months when the rainfall exceeds 15 mm during season t. This division is necessary because it is assumed in the model that runoff does not occur unless the monthly rainfall exceeds 15 mm. Therefore, for example, the amount of overland flow depends upon $PREC_{it}$, not TP_{it}.

The model assumes that the unit cost of supplying each user from a particular source is the same (in effect, equivalent to assuming that all the demands are concentrated at one location). Local effects, for example the severe drawdown at wells due to high pumping rates because of large single users, can be modelled by sharply increasing the cost of that source above a certain amount supplied.

This general schematic represents all the elements of the model; it is not required that every subarea have each of these elements.

Decision Variables

The following are the decision variables:

1. The amount of water taken from each source and the required capacity of that source.

2. The amount of water supplied to each consuming sector (net benefit maximization case only).

Constraint Set

A set of constraints is written for each season. The major constraints are listed below:

1. The total water demands in a subarea must be met.
2. Recycled water is limited by the amount of municipal and industrial water used.

3. Some demands can be satisfied only by certain sources (for example, during the wet season, a fraction of the nomadic herders and livestock demands must be supplied by ephemeral groundwater).

4. Some sources can only supply certain demands.

5. Only a fraction of the precipitation can be stored or used or recharged.

6. There must be continuity of surface water.

7. The annual amount of water pumped from aquifers of large storage capacity must be less than or equal to the annual recharge.

8. The seasonal amount of water pumped from aquifers of small storage capacity must be less than or equal to the seasonal recharge (both 7 and 8 can be modified so that either the water is mined or else used very conservatively.) Recharge to the aquifers can be both natural and artificial. There are two sources of artificial recharge. Method 1 is recharging aquifers with surface water, recycled water, or imported water. Method II is recharging with captured precipitation.

9. Only one reservoir operating policy can be selected per reservoir. Mixed integer programming is used in an attempt to capture the over year storage effects of reservoirs while only using a two season model, A 0-1 integer variable corresponds to a seasonal yield of a reservoir using a particular operating policy.

10. The supply sources and demands have upper and lower bounds (if necessary).

Objective Function

The objective function is either to minimize the costs of developing water supply sources to meet demands, or to maximize the net benefits of water use.

Detailed Discussion of Constraints

1. The total water demand in subarea i during season t must be met. Sources of supply include desalination ($DESAL_{it}$),* a fraction of the precipitation during t $\nu_i \cdot PREC_{it}$), the remainder of the water

*Variable list is at end of formulation in Table 2.4-1 imported from subarea j to i during t (a portion $(1-\alpha_{IM_{jit}})$ is lost to evaporation and seepage), withdrawn surface water (W_{it}), precipitation that has been captured on a large scale (ORS_{it}), precipitation that has been captured on a small scale $(ORSS_{it})$, recycled water $(RECY_{it})$, and ground water pumped from aquifer k (GW_{ikt}). The demands include demands for water use in i during t(D_{imt}), water exported from subarea i to j during t(IM_{ijt}), and artificial recharge by Method I to aquifer k during t(AR_{ikt}).

$$DESAL_{it} + \nu_i \cdot PREC_{it} + \sum_j \alpha_{IM_{ijt}} \cdot IM_{ijt} + W_{it} + ORS_{it} \tag{1}$$

$$+ ORSS_{it} + RECY_{it} + \sum_k GW_{ikt} \geq \sum_m D_{imt} + \sum_j IM_{ijt} + \sum_k AR_{ikt}$$

$\forall$ i, t

2. Recycled water cannot exceed its sources (fractions of industrial and municipal water).

$$RECY_{it} \leq \sum_k B_{ik} \cdot D_{ikt} \qquad \forall \; i, t \tag{2}$$

(k = appropriate municipal and industrial uses)

3. Recycled water can only supply a fixed proportion of the demands and water used in artificial recharge by Method I.

$$RECY_{it} \leq \sum_k \delta_{D_{ikt}} \cdot D_{ikt} + \sum_k \delta_{AR_{ikt}} \cdot AR_{ikt} \qquad \forall \; i, t \tag{3}$$

Constraints (2) and (3) are needed because recycled water cannot be used as a source everywhere. Recycled water can only be used where there are many users in a small area so the water can be economically collected, treated, and distributed.

4. Artificial recharge by Method 1 can only be supplied from surface water withdrawals, a proportion of the recycled water, and imported water.

$$\sum_k AR_{ikt} \leq W_{it} + \varepsilon_{ik} \cdot RECY_{it} + \Sigma\alpha_{IM_{jit}} IM_{ijt} \quad \forall i,t \qquad (4)$$

Only a portion of the recycled water can be used for this purpose because the source of recycled water may be far from aquifer k.

5. A fraction of some demands k' must be met by a fraction of some sources k.

$$\sum_k f_{ik} \cdot Source_{ikt} \geq \sum_{k'} s_{ik'} \cdot D_{ikt} \quad \forall i,t \qquad (5)$$

6. Some sources k can only satisfy a fraction of some demands k'

$$\sum_k Source_{jkt} \leq \sum_{k'} \gamma_{ik'} \quad D_{ikt'} \quad \forall i,t \qquad (6)$$

7. Only a fraction of the precipitation over a subarea can be controlled and used for artificial recharge by Method II to aquifer k (GWA_{ikt}), caught in unimproved or specially treated catchments for large scale storage (SSA_{it}) or else caught in small scale storage devices ($SSAS_{it}$).

$$\sum_k GWA_{ikt} + SSA_{it} + SSAS_{it} \leq \mu_i \cdot PREC_{it} \quad \forall i,t \qquad (7)$$

This constraint indicates that it is possible to capture and utilize only a portion of the rainfall. This is because of the uneven distribution of users, and physical limitations such as evapotranspiration.

8. The amount of precipitation caught in unimproved or specially treated catchments for large scale storage presently in storage ($SS_{i,t}$)

equals the remaining previously stored amount (a fraction $(1 - \alpha)$ is lost in evaporation) $(\alpha_{SS_{i,t-1}} \cdot SS_{i,t-1})$ plus the inflow from the catchment (SSA_{it}) minus the outflow from storage $(ORS_{i,t})$

$$\alpha_{SS_{i,t-1}} \cdot SS_{i,t-1} + SSA_{it} = SS_{it} + ORS_{it} \qquad \forall\ i,t \tag{8}$$

9. Constant (9) is the same as (8) except it is for precipitation caught in small scale storage devices.

$$\alpha_{SSS_{i,t-1}} \cdot SSS_{i,t-1} + SSAS_{it} = SSS_{it} + ORSS_{it} \qquad \forall\ i,t \tag{9}$$

10. The surface water volume or discharge at the critical point or center of the subarea (F_{it}) equals the inflow from upstream $(\sum_j I_{jit})$ plus overland flow entering the surface water $(\rho_i \cdot PREC_{it})$ plus tributary inflow above the critical point $(t_i \cdot TRIB_{it})$ plus yield from the reservoir $(\sum_k Y_{ikt})$ minus the fraction of the effluent flow lost to aquifer j upstream of the critical point or center $(\sum_j \pi_{ij} \cdot EF_{ijt})$ minus the surface water withdrawn (W_{it}).

$$\sum_j I_{jit} + \rho_i\ PREC_{it} + t_i TRIB_{it} + \sum_k Y_{ikt} - \sum_j \pi_{ij} \cdot EF_{ijt} - W_{it} = \tag{10}$$

$$F_{it} \quad \forall\ i,t$$

The term $\sum_k Y_{ikt}$ is the sum of the possible reservoir yields using different operating policies during t. Due to the 0-1 integer constraints used, only one of these yields can be positive and non-zero. The operating policies considered are:

1. no operation at all (i.e., natural flows).
2. constant yield year-round.
3. one third of yield in wet season, 2/3 of yield in dry season.

Only effluent flow from a body of surface water to an aquifer is modelled because in the area of the Sahel-Sudan studied in detail (Senegal, Mauritania), this is the only type of known surface water-aquifer flow. However, it is possible to model base flow using similar techniques.

11. The effluent flow is proportional to the flow in the center or at the critical point in a subarea.

$$EF_{ijt} = K_{ij} \cdot F_{it} \qquad \forall_{i,j,t} \tag{11}$$

This constraint is a simplification of the actual process. However, the only alternatives are either to use a highly non-linear constraint or a fairly complicated set of linear constraints. The second alternative would probably offer little improvement.

12. The flow leaving the subarea equals the remainder of the flow at the center or critical point after evaporation losses($o_i \cdot F_{it}$) plus overland flow and tributary flow entering downstream of the center or critical point flow ($\rho_i' \cdot PREC_{it} + (1-t_i)\ TRIB_{it}$) plus return flow from demands k ($\sum_k r_{ik} \cdot D_{ikt}$) minus effluent flow $\sum_j (1-\pi_{ij}) \cdot EF_{ijt}$) minus the amount of recycled water that comes from water uses that,without recycling would be returned to the surface water ($\xi_i \cdot RECY_{it}$).

$$\begin{aligned} & o_i \cdot F_{it} + \rho_i' \cdot PREC_{it} + (i-tl_i) \cdot TRIB_{it} + \sum_k r_{ik}\ D_{ikt} - \\ & \sum_j (1-\pi_{ij}) \cdot EF_{ijt} - \xi_i \cdot RECY_{it} = I_{ijt} \qquad \forall\ i,t \end{aligned} \tag{12}$$

The term $\xi_i \cdot RECY_{it}$ is necessary because some of the water usually returned to the surface water may be recycled in the final solution and hence not available to replenish the surface water.

13. <u>Groundwater Dynamics</u>

a. Aquifers of low storage capacity cannot provide overseason storage of recharge. They can receive recharge from effluent flow and/or a fraction of the total precipitation and/or a fraction of the sum of the monthly precipitation during t when the precipitation during a month exceeds 15 mm ($t_{ik} \cdot PREC_{it}$). A_{ik} is a coefficient that determines if the aquifer is to be operated on a safe yield basis ($A_{ik} = 1.0$), a conservative basis ($A_{ik} < 1.0$), or a mining basis ($A_{ik} > 1.0$).

$$GW_{ikt} \leq A_{ik} (\alpha_{EF_{ikt}} \cdot EF_{ikt} + t'_{ik} \cdot TP_{it} + t_{ik} \cdot PREC_{it}) \qquad (13a)$$

$$\forall \; i, t, k$$

b. Aquifers of large storage capacity can provide over-season storage (but not over year storage) of recharge. The recharge sources include those in 13a, as well as artificial recharge, return flow from water users, and run off from subareas j that enters the aquifer ($\sum_j RO_{jikt}$). A_{ik} has the same meaning as in 13a.

$$\sum_t GW_{ikt} \leq A_{ik} \sum_t (AR_{ikt} + \alpha_{GWA_{ikt}} \cdot GWA_{ikt} + \sum_j b_{ijk} \cdot D_{ijt}$$

$$-c_{ik} \cdot RECY_{it} + \sum_j RO_{jikt} + t_{ik} \cdot PREC_{it} + \alpha_{EF_{ikt}} \cdot EFikt + \qquad (13b)$$

$$t'_{ik} \cdot TPit) \qquad \forall_{i,k}$$

The term $c_{ik}RECY_{it}$ is the amount of user water that is normally returned to aquifer k but is now recycled. Both the total seasonal precipitation in a subarea (TP_{it}) and the sum of the monthly precipitation that exceeds 15 mm ($PREC_{it}$) is included in the recharge amount because oftentimes both contribute to an aquifer's recharge (see discussion of ground water recharge in Section 1.5.2.).

14. The runoff from subarea j to i that enters aquifer k during t equals a fraction of the sum of the monthly precipitation that exceeds 15 mm in j. This value of the precipitation is used because only this amount causes runoff.

$$RO_{jikt} = e_{jik} \cdot PREC_{jt} \qquad \forall\ i, t, k \qquad (14)$$

15. The design values of $PREC_{it}$, $TRIB_{it}$, TP_{it} are

$$\text{a.}\quad PREC_{it} = \overline{PREC}_{it} \qquad \forall\ i, t \qquad (15a)$$

$$\text{b.}\quad TRIB_{it} = \overline{TRIB}_{it} \qquad \forall\ i, t \qquad (15b)$$

$$\text{c.}\quad TP_{it} = \overline{TP}_{it} \qquad \forall\ i, t \qquad (15c)$$

16. The amount supplied from any source or stored in any facility or the yield from a reservoir has to be less than or equal to the source or facility capacity. In many cases, the units have to be converted for m^3/season to m^3/hr by a conversion factor (con_i).

a. Desalination

$$con_i \cdot DESAL_{it} \leq DESALM_i \qquad \forall\ i, t \qquad (16a)$$

b. Imports

$$con_i \cdot IM_{jit} \leq IMM_{ji} \qquad \forall\ i, t \qquad (16b)$$

c. Surface Water Withdrawal

$$con_i \cdot W_{it} \leq WM_{it} \qquad \forall\ i, t \qquad (16c)$$

d. Artificial Recharge by Method II

$$con_i \cdot GWA_{ikt} \leq GWAM_{ik} \qquad \forall\ i,k,t \qquad (16d)$$

e. Recycled Water

$$con_i \cdot RECY_{it} \leq RECYM_i \qquad \forall\ i,t \qquad (16e)$$

f. Groundwater from Aquifer k

$$con_i \cdot GW_{ikt} \leq GWM_{ik} \qquad \forall\ i,t,k \qquad (16f)$$

g. Artificial Recharge by Method I

$$con_i \cdot AR_{ikt} \leq ARM_{ik} \qquad \forall\ i,t,k \qquad (16g)$$

Constraints (16h) and (16i) are for the case of large scale precipitation catchment in unimproved or specially treated catchments and the storage of this precipitation in "farm ponds", behind small dams etc.

h. The Amount Stored

$$SS_{it} \leq SSM_{it} \qquad \forall\ i,t \qquad (16h)$$

i. The cost of the unimproved or specially treated catchments.

$$\frac{SSA_{it} \cdot AREA_i}{\sigma \cdot TP_{it}} \cdot C_{SSA_i} \leq CM_i \qquad \forall\ i,t \qquad (16i)$$

This constraint (16i) calculates the cost of the catchment by calculating the maximum area used in each season, multiplying that by the unit cost per area (C_{SSA_i}), and then selecting the largest cost that occurs (CM_i).

The maximum area is determined by dividing the volume caught (SSA_{it}) by the efficiency of catchment (σ_i) and the total season precipitation (TP_{it}) divided by the subarea area ($AREA_{it}$). This equation was derived by assuming that the maximum volume caught in any season is directly proportional to the maximum rainfall amount. Therefore if wTP_{it} is the maximum volume of rainfall in one storm ($w \leq 1.0$), then ω SSA_{it} is the maximum amount ever caught at one time. (Therefore, requiring the largest area). Thus, the maximum area needed in one season equals,

$$\frac{wSSA_{it} \cdot AREA_i}{\sigma_i \cdot wTP_{it}} = \frac{SSA_{it} \cdot AREA_i}{\sigma_i \cdot TP_{it}}$$

j. The amount of precipitation caught plus the previous amount in storage has to be less than or equal to the total storage capacity.

$$SSAS_{it} + \alpha_{SSS_i} \cdot SSS_{i,t-1} \leq SSSM_i \quad \forall\ i,t \quad (16j)$$

k. The seasonal yield from a reservoir using operating policy k

$$con_{ikt} \cdot Y_{ikt} \leq YM_{ik} \qquad \forall\ i,k \qquad (16k)$$

(con_{ikt} converts from seasonal yield to equivalent annual yield)

17. There is a maximum annual yield ($\overline{YM}_{ik}$) from any reservoir depending upon its operating policy k.

$$YM_{ik} \leq \overline{YM}_{ik} \cdot X_{ik} \qquad \forall\ i,\ k \qquad (17)$$

$\overline{YM}_{ik}$ is a constant . X_{ik} is a 0-1 integer variable. If k is the operating policy selected by the model, X_{ik} is 1. It is zero otherwise.

As set by the next constraints, only one X_{ik} can be equal to 1. Therefore, only one of the YM_{ik} can be non-zero and positive.

18. The reservoir in i can have only one operating policy.

$$\sum_k X_{ik} = 1 \qquad \forall\ i \qquad (18)$$

19. X_{ik} equals either 0 or 1.

20. All variables are greater than or equal to 0.

21. There are bounds on the values of many variables based upon political, social, economic, and physical feasibility. Most of the bounds are upper bounds except for those on F_{it} and I_{ijt} which are low flow constraints. Lower or fixed bounds can also be set on variables to model a source already existing or proposed.

$$ORS_{it} \geq \overline{ORS}_{it}$$

$$ORSS_{it} \geq \overline{ORSS}_{it}$$

$$D_{ikt} \leq = \geq \overline{D}_{ikt}$$

$$F_{it} \geq \overline{F}_{it}$$

$$I_{ijt} \geq \overline{I}_{ijt}$$

$$DESAL_{it} \leq \overline{DESAL}_{it}$$

$$RECY_{it} \leq \overline{RECY}_{it}$$

$$W_{it} \leq \overline{W}_{it}$$

$$GW_{ikt} \leq \overline{GW}_{ikt}$$

$$IM_{ijt} \leq \overline{IM}_{ijt}$$

$$AR_{ikt} \leq \overline{AR}_{ikt}$$

$$GWA_{ikt} \leq \overline{GWA}_{ikt}$$

$$DESALM_i \leq \overline{DESALM}_i$$
$$IMM_{ji} \leq \overline{IMM}_{ji}$$
$$WM_i \leq \overline{WM}_i$$
$$GWAM_{ik} \leq \overline{GWAM}_{ik}$$
$$RECYM_i \leq \overline{RECYM}_i$$
$$GWM_{ik} \leq \overline{GWM}_{ik}$$
$$ARM_{ik} \leq \overline{ARM}_{ik}$$
$$SSM_i \leq \overline{SSM}_i$$
$$SSSM_i \leq \overline{SSSM}_i$$
$$SSAM_i \leq \overline{SSAM}_i$$
$$SSASM_i \leq \overline{SSASM}_i$$
$$X_{ik} \leq 1 \quad \forall k$$

Objective Function

The objective function written below is the maximization of net benefits. OMR_{ik} and C_{ik} are the operation, maintenance and replacement costs and the capital cost respectively of source k in subarea i. $BNFT_i$ is the benefit associated with use ℓ in subarea i.

$$\text{Maximize } Z = -\sum_i\{\sum_t (OMR_{i,ORS_i} \cdot ORS_{it} + OMR_{i,ORSS_i} \cdot ORSS_{i,t}$$
$$+ OMR_{i,DESAL_i} \cdot DESAL_{it} + OMR_{i,RECY_i} \cdot RECY_{it} + OMR_{i,W_i} \cdot W_{it}$$
$$+ \sum_j OMR_{i,IM_{ji}} \cdot IM_{jit} + \sum_k OMR_{i,AR_{ik}} \cdot AR_{ikt} + \sum_k OMR_{i,GW_{ik}} \cdot GW_{ikt}$$
$$+ \sum_k OMR_{i,GWA_{ik}} \cdot GWA_{ikt} + OMR_{i,SS_i} \cdot SS_{it}$$

$$+ OMR_{i, SSS_i} \cdot SSS_{it} + OMR_{i, SSA_i} \cdot SSA_{it}$$

$$+ OMR_{i, SSAS_i} \cdot SSAS_{it} + \sum_e OMR_{i, Y_{ie}} \cdot Y_{iet})$$

$$+ C_{i, DESALM_i} \cdot DESALM_i$$

$$+ \sum C_{i, IMM_{ji}} \cdot IMM_{ji} + C_{i, WM_i} \cdot WM_i$$

$$+ \sum_k C_{i, GWAM_{ik}} \cdot GWAM_{ik} + C_{i, RECYM_i} \cdot RECYM_i$$

$$+ \sum_k C_{i, GWM_{ik}} \cdot GWM_{ik} + \sum_k C_{i, ARM_{ik}} \cdot ARM_{ik}$$

$$+ C_{i, SSM_i} \cdot SSM_i + C_{i, SSSM_i} \cdot SSSM_i$$

$$+ CM_i + C_{i, SSAM_i} \cdot SSASM_i$$

$$+ \sum_e C_{i, YM_{ie}} \cdot YM_{ie} + \sum_e OMR_{i, YM_{ie}} \cdot YM_{ie} \}$$

$$+ \sum\sum\sum_{iet} BNFT_{ie} \cdot D_{iet}$$

Should it be desirable to consider distribution costs explicitly, the term $-\sum\sum\sum_{iet} U_{iet} D_{iet}$ which relates distribution costs to users can be added to be ɔjective function. U_{iet} is the distribution cost of user e in subarea i during t. D_{iet} is the demand of use e in subarea i during t. In a least cost formulation (demands fixed at prespecified levels), the term $\sum\sum\sum_{iet} U_{iet} D_{iet}$ is a constant and thus need not be included in the cost minimization process.

TABLE 2.4-1

Definitions of Variables and Coefficients

Variables

AR_{ikt}	=	Amount of water artificially recharged by Method I to aquifer k in i during t (Method I is recharge of surface water, recycled water, or imported water).
$AREA_i$	=	Area of subarea.
ARM_{ik}	=	Capacity of artificial recharge facility by Method I in subarea i to aquifer k.
$C_{SSA,i}$	=	Unit cost of unimproved or specially treated catchment in i.
CM_i	=	Cost of unimproved or specially treated catchment in i.
D_{ikt}	=	Demand of type k in i during t.
$DESAL_{it}$	=	Amount of water supplied from desalination in i during t.
$DESALM_i$	=	Capacity of desalination plant in i.
EF_{ikt}	=	Effluent flow from surface water to aquifer k in i during t.
F_{it}	=	Surface water flow at critical point in i during t.
GW_{ikt}	=	Amount of water pumped from aquifer k in i during t.
GWA_{ikt}	=	Amount of water artificially recharged to aquifer k in i during t by Method II (Method II is recharge of collected runoff).
$GWAM_{ik}$	=	Capacity of artificial recharge facility to aquifer k by Method II in i.
GWM_{ik}	=	Capacity of groundwater pumping facility from aquifer k in i.
I_{jit}	=	Surface water entering i from j during t (usually stream flow).

IM_{jit} = Amount of water imported into i from j during t.

ORS_{it} = Amount of water taken from large scale rainfall and runoft storage for demand usage in i during t.

$ORSS_{it}$ = Amount of water taken from small scale storage of precipitation in i during t.

$PREC_{it}$ = Sum of monthly precipitation in i that exceeds 15 mm per month during t.

$RECY_{it}$ = Amount of water recycled in i during t.

$RECYM_i$ = Capacity of recycling plant in i.

RO_{ijkt} = Runoff from i to j that recharges aquifer k in i during t.

SS_{it} = Amount of runoff or precipitation in large scale storage in i during t.

SSA_{it} = Amount of precipitation caught in improved catchments in i during t for large scale storage.

$SSAM_i$ = Capacity of improved catchments in i.

$SSAS_{it}$ = Amount of precipitation caught in i during t for small scale storage.

SSM_i = Capacity of large scale runoff and precipitation storage facilities in i.

$SSSM_i$ = Capacity of small scale precipitation storage facilities in i.

TP_{it} = Total amount of precipitation in i during t.

$TRIB_{it}$ = Amount of tributary inflow into i during t.

U_{iet} = Distribution cost of use e in subarea i during t.

W_{it} = Amount of surface water withdrawn in i during t.

WM_{it} = Capacity of surface water withdrawal facility in i.

X_{ik} = 0-1 integer variable corresponding to operating policy k for reservoir in i.

Y_{ikt} = Yield from reservoir in i using operating policy k during t.

YM_{ik} = Annual yield from reservoir in i using operating policy k.

All variables with a line over them are upper bounds of their respective variables or are design quantities.

Coefficients

A_{ik}	=	Proportion of recharge to aquifer k in i that can be pumped.
b_{ijk}	=	Proportion of water use j in i that is returned to aquifer k.
B_{ij}	=	Fraction of water demand of type j that can be recycled in i.
c_{ik}	=	Proportion of recycled water in i that is taken from non-consumed demand water that is usually recharged to aquifer k.
con_i	=	Conversion factor from seasonal use to capacity needed in i.
e_{jik}	=	Fraction of precipitation in j that recharges aquifer k in i.
E_{ik}	=	Fraction of recycled water that can be recharged to aquifer k by Method 1.
f_{ik}	=	Fraction of source k in i that has to be supplied to a demand k' in i.
K_{ij}	=	Fraction of surface water at critical point that flows into aquifer j in i.
o_i	=	Fraction of flow at critical point left after evaporation and seepage losses.
r_{ik}	=	Fraction of demand water of type k in i that is returned to surface water after use.
s'_{ik}	=	Fraction of demand in i of type k that must be satisfied by a particular set of sources.
t_i	=	Fraction of tributary inflow in i that enters upstream or near critical point in i.
t_{ik}	=	Fraction of precipitation ($PREC_{it}$) that is recharged to aquifer k in i.
t'_{ik}	=	Fraction of total precipitation (TP_{it}) that is recharged to aquifer k in i.

α_i = Fraction of water of source i that is left after evaporation and seepage losses.

γ_{ik}' = Fraction of demand k' in i that can be satisfied by a set of sources k in i.

δ_{ij} = Fraction of demand of type j that can be satisfied by recycled water in i.

ε_{ik} = Fraction of water artificially recharged via Method I to aquifer k in i that can come from recycled water.

μi = Fraction of precipitation that can be either artificially recharged using Method II or caught for either large scale or small scale storage.

ν_i = Fraction of precipitation that can be used immediately to supply demand at no cost

ξ_i = Fraction of recycled water in i that is taken from nonconsumed demand water that usually returns to the surface water.

π_{ij} = Fraction of effluent flow to aquifer j in i lost from surface water upstream of, or near critical point.

ρ_i = Fraction of precipitation (probably in form of runoff) entering surface water upstream of, or near critical point in i.

ρ_i' = Fraction of precipitation entering surface water downstream of critical point in i.

σ = Efficiency of unimproved or specially treated catchments in i.

2.5 Illustrative Applications of the Planning Framework: Applications to Senegal and Mauritania as Case Studies

The purpose of the case studies is to show how the supply and demand models can be applied to the six Sahel-Sudan countries, and how these models are used in the planning process. The countries of Mauritania and Senegal were chosen for the case study. The choice was made primarily because of data availability for these two countries.

Least cost runs and a net benefit maximizing run were made. The basis of the least cost runs was a set of 1990 water demands that emphasized a large amount of agricultural development, particularly of crops. The net benefit run was primarily concerned with determining what benefits must accrue to these uses such that the quantities of water supplied are justified. In both types of runs it was generally assumed relatively advanced methods and labor rates would be used in developing all the sources except the dams for storing surface runoff for which it was assumed local methods would be used.

Two seasons were modelled, the wet season (June through October) and the dry season (November through May).

2.5.1. Subarea division: The two countries were divided into subareas according to the criteria described in Section 2.2. Mauritania has 9 subareas, and Senegal has 6 subareas. The subareas are shown in Figures 2.5-1 and 2.5-2. Subareas M8 and M7 of Mauritania cover part of Senegal because it is difficult to model a situation where the banks of a river are in different countries. Other than this exception, areas do not cross national boundaries.

2.5.2. Supply sources: The sources of supply available for each subarea are described in Table 2.5-1. The information sources on the locations of groundwater were U.N. (1973), Archambault (1960), Bourguet (1966), Jones (1974), Depagne and Moussou (1969), Paloc (1962),

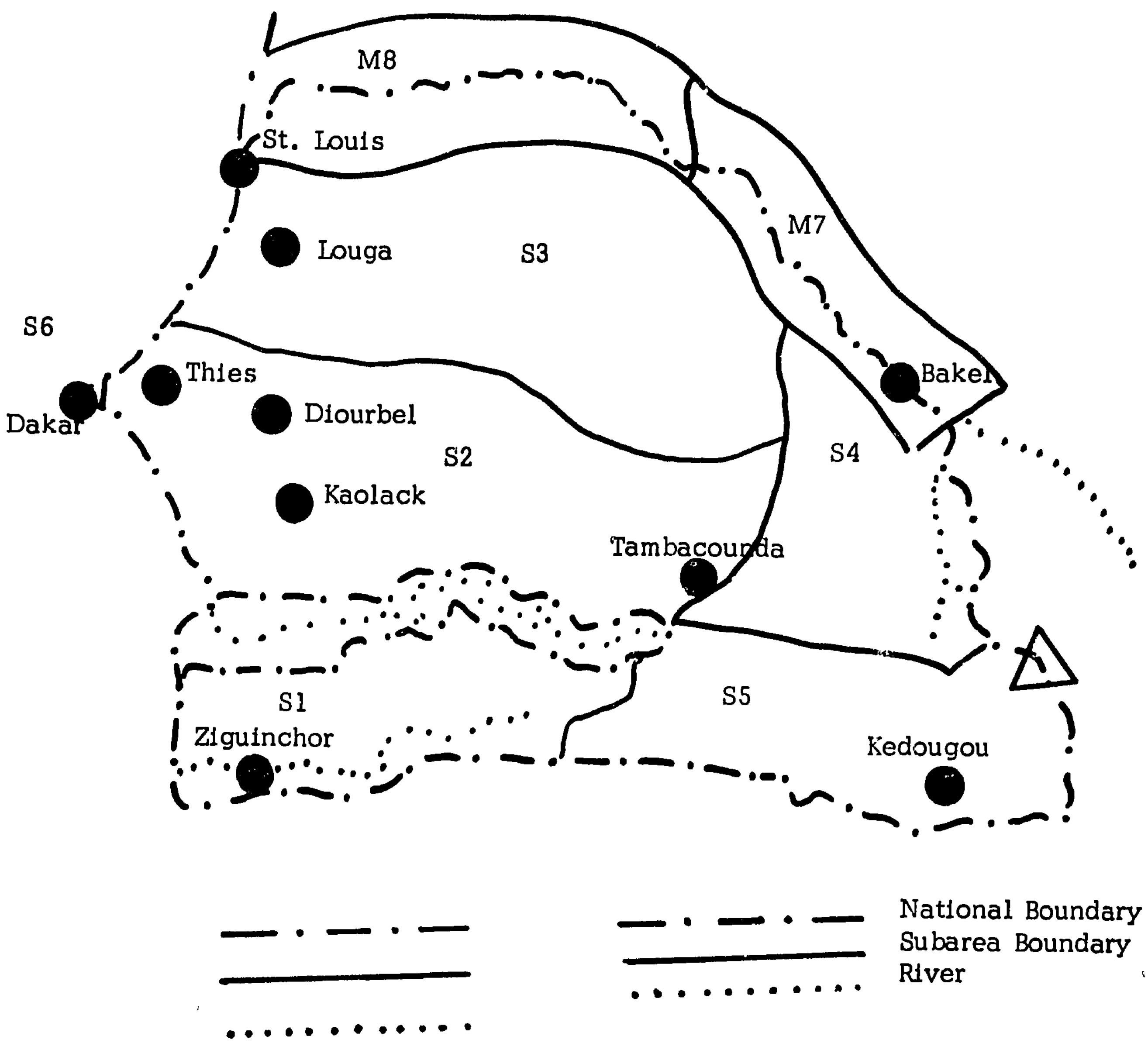

FIGURE 2.5-1 Subareas of Senegal

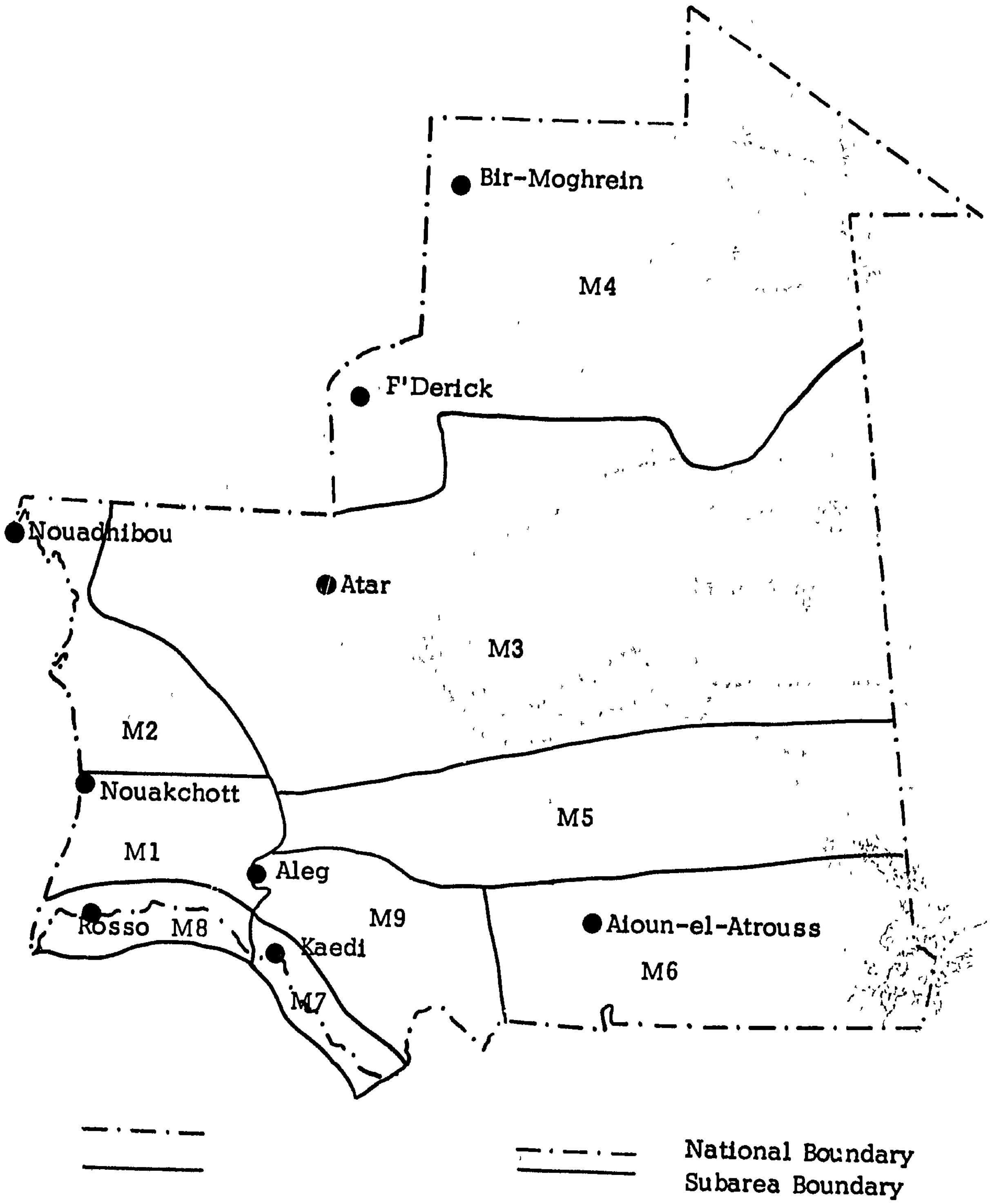

FIGURE 2.5-2 Subareas of Mauritania

and Pallas (1972). Jones' estimates of the locations and quantities of Nubian sandstone groundwater are preliminary, unpublished, estimates subject to revision. As noted in Table 2.5-1, some subareas have only small scale storage of runoff and precipitation. The reason is that for large amounts of runoff to occur, the land surface must be fairly impermeable. It was judged that only subareas with mostly rock aquifers had fairly impermeable surfaces, and hence could have large scale storage.

A source is described as local in Table 2.5-1 if it cannot be considered fairly evenly distributed throughout the subarea. In most cases a local source is one located along or near the border of a subarea. To account for the greater expense of its use, the cost of piping the water from this source to the center of the subarea is added to the normal costs of the source. The critical point referred to in the description of surface sources is the point where the surface water is withdrawn. There is a low flow constraint at the critical point to account for navigation, hydroelectric power, and pollution control needs. The major rivers in a subarea are modelled explicitly; minor rivers are modelled as runoff input and do not have low flow constraints associated with them.

Existing water transfers and those that appear to have reasonably possible advantages were modelled. The same principles were followed in selecting possible desalination sites.

2.5.3. Low flow constraints: The low flow values at both the critical point and outflow point of M8 were set at 100 m^3/sec. As discussed in Section 1.4.1, 100 m^3/sec. provides enough water for salinity control, and the navigation of 1.4 m draft vessels. The low flow values at the corresponding points in M7 were set at 64 m^3/sec., the minimum for the navigation of 1.4 m draft vessels. These flows

(text continued on page 191)

TABLE 2.5-i

Sources of Water Supply

Subarea M 1

Groundwater

Aquifer #	Aquifer Type	Comments
2	Continental Terminal	Located in western part of subarea, no recharge, artificially recharged by Method I (maximum annual withdrawal is $10 \times 10^6 m^3$).
3	Ephemeral	
5	Continental Terminal	Located in extreme eastern part of subarea, local source, recharged by runoff from M9.

Surface Water

None

Imports From M8

Other Sources (✓ = Yes)

Desalination	✓
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	
Recycling	✓

Sources of Water Supply

Subarea M2

Groundwater

Aquifer #	Aquifer Type	Comments
2	Continental Terminal	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)
3	Ephemeral	

Surface Water

None

Imports From None

Other Sources (✓= Yes)

Desalination	✓
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	
Recycling	✓

Sources of Water Supply

Subarea M3

Groundwater Aquifer #	Aquifer Type	Comments
1	Rock	Artifically recharged by Method I
2	Sand	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)
3	Ephemeral	
4	Nubian Sandstone	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)

Surface Water

None

Imports From None

Other Sources (✓= Yes)

Desalination
Small Scale Storage of Rainfall and Runoff ✓
Large Scale Storage of Rainfall and Runoff ✓
Recycling ✓

Sources of Water Supply

Subarea M4

Groundwater

Aquifer #	Aquifer Type	Comments
1	Rock	No recharge
2	Continental Terminal	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)
3	Ephemeral	No recharge
4	Nubian Sandstone	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)

Surface Water

None

Imports From None

Other Sources (✓= Yes)

Desalination
Small Scale Storage of Rainfall and Runoff ✓
Large Scale Storage of Rainfall and Runoff ✓
Recycling ✓

Sources of Water Supply

Subarea M5

Groundwater Aquifer #	Aquifer Type	Comments
1	Sand	
2	Rock	
3	Ephemeral	

Surface Water

None

Imports From None

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	✓
Recycling	✓

Sources of Water Supply

Subarea M6

Groundwater

Aquifer #	Aquifer Type	Comments
1	Rock	
2	Continental Terminal	Local source
3	Ephemeral	
4	Sand	

Surface Water

None

Imports From None

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	✓
Recycling	✓

Sources of Water Supply

Subarea M7

Groundwater

Aquifer #	Aquifer Type	Comments
1	Rock	
2	Maestrichtian	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)
3	Ephemeral	

Surface Water

Senegal River (critical point is Matam)

Imports From

None

Other Sources (√= Yes)

Desalination
Small Scale Storage of Rainfall and Runoff √
Large Scale Storage of Rainfall and Runoff √
Recycling √

Sources of Water Supply

Subarea M8

Groundwater

Aquifer #	Aquifer Type	Comments
1	Alluvial	
2	Ephemeral	

Surface Water

Senegal River (critical point is Podor)

Imports From None

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	
Recycling	✓

Sources of Water Supply

Subarea M9

Groundwater

Aquifer #	Aquifer Type	Comments
1	Rock	Artifically Recharged by Method II
2	Sand	
3	Ephemeral	

Surface Water

Runoff from Gorgol is in $\nu_1 \cdot PREC_{1t}$ term

Imports From

M7

Other Sources (✓ = Yes)

Source	
Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	✓
Recycling	✓

Sources of Water Supply

Subarea S1

Groundwater

Aquifer #	Aquifer Type	Comments
2	Continental Terminal	Artifically recharged by Method I, recharge rate is 20% of rainfall
3	Ephemeral	

Surface Water

Casamance River (critical point is Sedhiou)

Imports From None

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	
Recycling	✓

Sources of Water Supply

Subarea S2

Groundwater

Aquifer #	Aquifer Type	Comments
1	Continental Terminal	
3	Ephemeral	
4	Maestrichtian	No recharge (annual maximum withdrawal is $10 \times 10^6 m^3$)

Surface Water

None

Imports From None

Other Sources (✓ = Yes)

Desalination
Small Scale Storage of Rainfall and Runoff ✓
Large Scale Storage of Rainfall and Runoff
Recycling ✓

Sources of Water Supply

Subarea S3

Groundwater

Aquifer #	Aquifer Type	Comments
1	Continental Terminal	
3	Ephemeral	
4	Maestrichtian	No recharge (maximum annual withdrawal is $10 \times 10^6 m^3$)

Surface Water

None

Imports From

M8

Other Sources (√ = Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	√
Large Scale Storage of Rainfall and Runoff	
Recycling	√

Sources of Water Supply

Subarea S4

Groundwater Aquifer #	Aquifer Type	Comments
1	Rock	
2	Continental Terminal	Local source
3	Ephemeral	

Surface Water

Falémé River (critical point is confluence of Falémé and Sanokole Rivers)
Grandamaka River (tributary)

Imports From

M7

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	✓
Recycling	✓

Sources of Water Supply

Subarea S5

Groundwater

Aquifer #	Aquifer Type	Comments
1	Rock	
2	Continental Terminal	Local source
3	Ephemeral	

Surface Water

Falémé River (critical point is immediately downstream of Gourbassi Dam site)

Imports From

Other Sources (✓= Yes)

Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	✓
Recycling	✓

Sources of Water Supply

Subarea S6

Groundwater Aquifer #	Aquifer Type	Comments
1	Alluvial	Because of urbanization effects, it was assumed that only 50% of the total area of the subarea was available for groundwater recharge.
3	Ephemeral	

Surface Water

None

Imports From S2, S3, M8

Other Sources (✓= Yes)

Source	
Desalination	
Small Scale Storage of Rainfall and Runoff	✓
Large Scale Storage of Rainfall and Runoff	
Recycling	✓

Note: It was assumed that because of urbanization effects, only 50% of the area of the subarea was suitable for direct use of precipitation. The amount of precipitation available for small scale storage was also decreased 50%.

should also assure that large quantities of hydroelectric power are produced by both dam power plants and run-of-the-river plants.

The low flow value set for the Casamance River in S1 was 3 m^3/sec. The average flow rate is approximately 7 m^3/sec. It was assumed that 3 m^3/sec. would provide enough flow for navigation as well as for fishing. No low flow values were set for the Falémé River in S4 and S5.

2.5.4. Cost of supply: Shown in Table 2.5-2 are the costs of using the alternative sources of water. An operation period of 50 years was assumed for each source. If the lifetime of the equipment was less than fifty years (i.e., twenty-five years), a new installation after the first twenty-five years was assumed. All costs were discounted, using an interest rate of ten percent. The 50 year discounted operation, maintenance, and replacement (OMR) cost in Table 2.5-2 is the total OMR cost per cubic meter of water taken from the source every year for fifty years. Most of the equipment (for example, pumps) was assumed to operate at seventy percent efficiency. The procedure to determine most costs was to determine the cost in the United States and then increase that value by 30 percent to allow for added transportation and other costs. All costs were adjusted to be those of June, 1974 by using the Engineering News (1974) Construction Cost Index of June 20, 1974.

As discussed in Section 2.4.3, in the least cost formulation of the supply model, the distribution costs are not explicitly considered in the objective function; they are constant given the demand level.

All costs were assumed to be linear. They were linearized within the range of interest. The effect of the unit costs selected upon the solution can be determined using sensitivity analysis.

The cost of using groundwater was based upon the information

TABLE 2.5-2

Water Supply Cost

( ) = Cost if not recharged

Source	Capital cost ($/m^3/hr.)	Total 50 year discounted operation, maintenance, replacement cost ($/m^3)
Alluvial Aquifier	263	.00778
Continental Terminal Aquifer	878 (1,752)	.0456(.0619)
Ephemeral Aquifer	4,204	.0862
Maestrichtian Aquifer	(3,416)	(.0982)
Nubian Sandstone Aquifer	(19,360)	(.4015)
Rock Aquifer	29,871 (59,742)	
Sand Aquifer	3,504 (7,008)	
Desalination	24,800	
Artificial Recharge Basins	180	.0667
Improved Catchments For Increased Runoff	0.0	0.0
Large Scale Storage of Rainfall and Runoff	.000104/m^3	.000002
Small Scale Storage of Rainfall	.00104/m^3	.00002
Recycling	10,400	.54
Pumping Water from Surface Water (assuming 30 ft. head)	78	.00987
Transferring Water Between Subareas	170 m^3/hr-km	Replacement & Maintenance 4.16/m^3/hr-km; Operation, .0015/m^3-km
Reservoirs	See Section 2.5.5	

given in Section 1.5.4. It was assumed that all wells in a given formation produced the same yield. The cost of connecting pipe in well fields was ignored because extensive well fields are generally not a possibility in the Sahel-Sudan region as groundwater users are usually far apart.

The cost of desalination was based upon U.N. (1964). This report contained 1959 cost information for desalination in Nouakchott, Mauritania. The OMR cost was reported to be .2 percent of the capital cost in addition to fuel costs. It was estimated that fuel would be available to a desalination plant at the price of $.14/liter.

Todd (1964) was the source of data for estimating artificial groundwater recharge costs. It was assumed that the recharge was done via recharge basins.

No costs were assigned to the catchments because it was assumed they were used in their natural states. However, Clüff and Dutt (1973) provide cost data on specially treated catchments.

No cost data was available on the small dams ("petits barrages") used to store surface runoff. It was assumed that such dams would cost 1 percent the construction cost of large dams in Mali. The costs of providing small scale storage in cisterns, small tanks, etc., were assumed to be ten times the cost of the small dams. It is entirely possible that both these costs are less.

It was assumed that the cost of recycling water would be equivalent to that of providing secondary waste treatment. Tertiary treatment costs were not used because it was assumed that not all the recycled water would be used for human use and that many urban areas would have adequate treatment facilities in their distribution systems. Data on the cost of secondary treatment was taken from Fair, Geyer and Okun (1966).

It was assumed that a thirty foot head difference would have to be overcome to pump surface water to users' distribution systems. Cost data were taken from Charles T. Maguire and Associates (1967). Fuel costs were assumed to be $.14/liter.

The costs of long distance transportation of water were based upon pipeline and pump station costs given in Cederstrom (1973) and a fuel cost of $.14/liter. It was assumed that there was no elevation difference between the source and the user. The head loss was due to pipeline resistance only.

2.5.5. Reservoir sites, operation, and costs. The case study considers two reservoir "sites". The first one is the Gourbassi Dam site on the Falémé River in Subarea S5. The average annual flow at this site is reported to be 165 m^3/sec. (Senegal-Consult 1970). The other site is assumed to exist in Subarea M7 and to control the Senegal River and all its tributaries upstream of Bakel except the Falémé. (The actual site is not in M7, but rather would be most likely composed of the Manantali and Galougo sites in Mali.) The average annual flow at this site (minus the contributions from the Falémé River) is 605 m^3/sec. (Senegal-Consult 1970).

Every site has three possible operating policies. The first was that of no operation (i.e., the dam did not exist at all, and natural flows prevailed). The second policy was constant yield throughout the year. The final policy was one third of the yield in the wet season and two-thirds in the dry season (1/3 - 2/3 operating policy). It was assumed that the maximum annual volume of yield available at each site under each operating policy was the present annual average flow volume minus 10% to account for evaporation loss.

The costs of the dams were based upon data in Senegal-Consult (1970) for the Galougo, Gourbassi, and Manantali Dam sites. For the

TABLE 2.5-3

Yields and Costs of Reservoirs

Site Location	Operating Policy	Maximum Yield ($m^3 \times 10^6$) Wet Season	Dry Season	Cost of Annual Yield ($\$10^6/10^6\ m^3$) Capital	OMR
M7	Constant Yield	7,148	10,008	.0199	.00074
M7	1/3 - 2/3	5,719	11,437	.0249	.00093
S5	Constant Yield	1,945	2,722	.0119	.00042
S5	1/3 -2/3	1,556	3,111	.0149	.00053

constant yield operating policy, the unit cost of the site in M7 was determined by dividing the total cost of the Galougo and the Manantali by the total yield. The unit cost at the site in S5 was done in a similar manner except using the Gourbassi data. For the 1/3 - 2/3 operating policy, the previous costs were increased by 25% to account for the additional storage needed. The values are in Table 2.5-3.

2.5.6. Coefficients of variables: Shown in Table 2.5-4 are the coefficients of the variables. Most of the values were estimates based upon what appeared reasonable. Little hydrologic or social data wereavailable to really improve them. However, the sensitivity of the solutions to some of them was investigated.

The value of K_{ij} in Equation 11 of the formulation was estimated only for subarea M8 because this was judged to be the only subarea where there was significant groundwater - surface water interaction. Only effluent flow was modelled, because the average water table height of the aquifer bordering the Senegal River is always significantly less than the height of the water surface of the river. The basis of the calculation of K_{ij} was that the 1960-1964 annual average recharge to the bordering aquifer is about $120 \times 10^6 m^3$ (Depagne and Gouzes 1969), and that 730 m^3/sec. is needed at Bakel to cause flooding in the valley. These figures then were used to plot a graph of recharge versus seasonal flow volume which was linearized. This is an approximation. Its influence on the solution can be checked with sensitivity analysis.

2.5.7. Hydrologic input: The initial run of the case study used average values of rainfall and surface water flow. The sources of monthly rainfall values were ASECNA (1969) and Organization of African Unity (1967). In most cases, the average monthly rainfall in a subarea was based upon one or two stations. Sources of surface (text continues on page 200).

TABLE 2.5-4

Model Constants for Example Problem

Constant	Equation Number in Formulation	Value	Comments
ν_i	1	.01	This means 1% of the seasonal precipitation causing runoff is available to meet the total demand. This water is not stored and is used immediately.
$\alpha_{Im_{ijt}}$	1, 4	1.0	There is no evaporation or seepage loss of transferred water because it is piped.
B_{ik}	2	.2-.3	Generally the only sources of recycled water are human or industrial users.
δ_{ik}	3	.2-.3	Generally the only users of recycled water are human, industrial or irrigation users.
$\delta_{AR_{ikt}}$	3	1.0	Artificial recharge by Method I usually occurs near a population center and therefore can use all available recycled water.
ε_{ik}	4	1.0	Same as above.
f_{ik}	5	1.0	Generally, this constraint was only written for Mauritania and required that 30% of the human and livestock demands during the wet season must be met by ephemeral groundwater This is to model the fact that presently during the wet season, a large proportion of the population relies upon the ephemeral groundwater.
$s_{ik'}$	5	.3	

Table 2.5-4 (continued)

Constant	Equation Number in Formulation	Value	Comments
γ_{ik}	6	.1	This constraint is used to require that desalination in subareas M1 and M2 can supply at most 10 percent of the human population.
μ_i	7	.01	This means it is only possible to utilize 1 percent of the precipitation for artificial recharge by Method II, improved catchment, or small scale storage.
$\alpha\ SS_{i,t-1}$	8	.5, .8	After the wet season, 80 percent of the stored amount is available. After the dry season, 50 percent of the stored amount is available.
$\alpha\ SSS_{i,t-1}$	9	.5, .8	Same as above.
ρ_i, t_i, π_{ij}	10		For M8, ρ_i, t_i are 0.0 as there are no significant inflows in this subarea. For M7, t_i is 0.0, $_i$ is .025, because 25 percent of M7 contributes runoff to this portion of the Senegal River. The runoff coefficient is 10 percent. π_{ij} is 0.0 in M7 indicating no groundwater-surface water interaction. π_{ij} is .5 in M8 indicating one half of the effluent flow is lost in this portion of M8. In S1, ρ_i is .005 because it is assumed that the runoff coefficient here is only 1 percent (because of vegetation), and half enters this portion of the river. t_i and π_{ij} are 0.0. In S4, π_i is .05 because

(cont.)

Table 2.5-4 (continued)

Constant	Equation Number in Formulation	Value	Comments
			it is estimated that only 50 percent of the area of S4 contributes runoff to the river here (runoff coefficient equals 10 percent). t_i is 1.0. Most of the tributary inflow is from the Grandamaka River, Π_{ij} is o.o. In S5, ρ_i is .01 because it is assumed 10 percent of the area contributes runoff (runoff coefficient equals 10 percent). t_i and Π_{ij} are 0.0.
K_{ij}	11	.00270, .00250	(See discussion in text, Section 2.5.6.)
O_i	12	.8	
ρ'_i	12		In M8, M7, S1, S5, ρ'_i is 0.0. In S4, $\rho'_i = \rho_i$
r_{ik}, ξ_i	12	0.0	
	13		See description of groundwater recharge in Section 1.5.2.
e_{jik}	14	.05	It is reported by Bourguet (1966) that the aquifers of the eastern part of M1 are recharged by runoff from M9. It was estimated that 50 percent of the runoff from M9 supplies this recharge.

water data were Senegal-Consult (1970) and Chaperon (1974). It was assumed that the maximum annual volume of yield available at each reservoir site under each operating policy was the present annual average flow volume minus 10% to account for evaporation loss.

The later runs of the model used lower values of rainfall and surface water flow to simulate the effects of a major drought. For these runs, total precipitation (TP_{it}) values were 30 percent less, total monthly precipitation exceeding 15 mm. per month ($PREC_{it}$) values were 50 percent less, tributary inflows ($TRIB_{it}$) were 50 percent less. The reservoir yields were assumed 10 percent less. This seemed appropriate because Senegal-Consult (1970) found, using simulation techniques, that if approximately 90 percent of the average flow values were used as yields, flow deficits occurred with a probability of 0.10. It would be during a drought period that such deficits would occur. Therefore, to use the 90 percent of average flow as yields would be erroneous. Consequently, the yields were decreased an additional 10 percent to provide a reasonable "guaranteed" yield during a drought period .

The selection of hydrologic design conditions is discussed in detail in Section 3.2.

2.5.8 Demand data: Shown in Tables 2.5-5 and 2.5-6 are the water demand quantities used in the initial run of the supply model. The majority of the human, industrial, and livestock demands are based upon demand projections made by the demand model for the year 1990 emphasizing rural development. However, some of the values have been adjusted.

It is assumed that the population is entirely sedentary. However, a nomadic population could be modelled by changing the seasonal demands.

Table 2.5-5

Senegal Water Demands by Use, by Subarea

1990

Favoring Ruralization (*=adjusted value)

Sub area	Demand Type	Demand Quantity	Use Coefficient	Water Use	Wet season use m3	Dry season use m3	Comments
S1	Human	$.790\times10^{6}$ persons	30.0 l/day/persn	23.7×10^{6} l/day	3.55×10^{6}	4.98×10^{6}	
	Industrial	16 (numbers refer to Table_____)		29.50×10^{6} l/yr*	$.01\times10^{6}$	$.02\times10^{6}$	5.90×10^{6} KWH/yr
	Crop	50,000 ha		$90\ 7\times10^{9}$ l/yr	648×10^{6}	260×10^{6}	April through October
	Livestock	$.312\times10^{6}$ amu	66.7 l/day/	20.81×10^{6} l/day	3.12×10^{6}	4.37×10^{6}	
S2	Human	$.445\times10^{6}$ persons	30.0 l/day/persn	13.36×10^{6} l/day*	2.01×10^{6}	2.81×10^{6}	
	Industrial	3,4,5,12,16,17		$270{,}300\times10^{6}$ l/yr*	112.1×10^{6}	158.2×10^{6}	1.00×10^{6} tons/yr of fertilizer 5.90×10^{6} KWH/yr of energy $.331\times10^{6}$ tons/yr of cement
	Crop	25,000 ha (May thru Sept.)		322×10^{9} l/yr	260×10^{6}	60×10^{6}	
	Livestock	$.643\times10^{6}$ amu	66.7 l/day/amu	42.89×10^{6} l/day	6.43×10^{6}	9.01×10^{6}	
S3	Human	$.76\times10^{6}$ persons	30.0 l/day/persn	22.83×10^{6} l/day	3.4×10^{6}	4.79×10^{6}	
	Industrial	16,		29.50×10^{6} l/yr*	$.01\times10^{6}$	$.02\times10^{6}$	5.90×10^{6} KWH/yr of energy
	Crop	10,000 ha		129×10^{9} l/yr	100.00×10^{6}	30×10^{6}	May through September
	Livestock	3.07×10^{6} amu	66.7 l/day/amu	205.06×10^{6} l/day	30.76×10^{6}	43.06×10^{6}	
S4	Human	$.310\times10^{6}$ people	30.0 l/day/persn	9.30×10^{6} l/day	1.39×10^{6}	1.96×10^{6}	
	Industrial	16,		14.75×10^{6} l/yr	$.01\times10^{6}$	$.01\times10^{6}$	2.95×10^{6} KWH/yr of energy
	Crop	50,000 ha		644×10^{9} l/yr	520×10^{6}	130×10^{6}	May through September
	Livestock	1.14×10^{6} amu	66.7 l/day/amu	76.04×10^{6} l/day	11.41×10^{6}	15.97×10^{6}	
S5	Human	$.38\ 8\times10^{6}$ people	30.0 l/day/persn	28.19×10^{6} l/day	1.80×10^{6}	2.50×10^{6}	
	Industrial	16,		7.38×10^{6} l/yr	0.0	0.0	1.48×10^{6} KWH of energy/yr.
	Crop	50,000 ha		907×10^{9} l/yr	640×10^{6}	260×10^{6}	April through October
	Livestock	1.72×10^{6} amu	66.7 l/day/amu	114.72×10^{6} l/day	17.21×10^{6}	24.09×10^{6}	
S6	Human	1.434×10^{6}	30.0 l/day/persn	43.00×10^{6} l/day	6.45×10^{6}	9.03×10^{6}	
	Industrial	16,17		$700{,}000.00\times10^{6}$ l/yr	291.67×10^{6}	408.33×10^{6}	2.59×10^{6} ton/yr of fertilizer 2.59×10^{6} KWH/yr of energy
	Crop						
	Livestock	0.0	66.7 l/day/				

Table 2-5.6
Mauritania Water Demands by Use, by Subarea
1990
Favoring Ruralization (*=adjusted value)
Permanent (no migration)

Sub area	Demand Type	Demand Quantity	Use Coefficient	Water Use	Wet Season Use (June-Oct) (m3)	Dry Season Use (Nov-May)(m3)	Comments
M 1	Human	$.410\times10^{6}$* people		12.30×10^{6} l/dy	1.84×10^{6}	2.58×10^{6}	11,934 tons/yr of Copper
	Industrial	15,16 (numbers refer to Table__)		51.75×10^{6} l/yr	$.02\times10^{6}$	$.03\times10^{6}$	2.95×10^{6} /yr of energy
	Crops	10,000 ha		129×10^{9} l/yr	100×10^{6}	30×10^{6}	May through September
	Livestock	0.0		0.0			
M 2	Human	$.040\times10^{6}$ people		1.20×10^{6} l/dy	$.18\times10^{6}$	$.26\times10^{6}$	
	Industrial	5,16		1565.82×10^{6} l/yr	$.65\times10^{6}$	$.91\times10^{6}$	.0183 18×10^{6} KWH/yr of energy
	Crops						
	Livestock	$.053\times10^{6}$ amu		3.53×10^{6} l/dy	$.53\times10^{6}$	$.74\times10^{6}$	
M 3	Human	$.26\times10^{6}$ people		7.76×10^{6} l/dy	1.16×10^{6}	1.63×10^{6}	
	Industrial			0.0	-		
	Crops						
	Livestock	$.954\times10^{6}$ amu		24.73×10^{6} l/dy	3.71×10^{6}	5.19×10^{6}	
M 4	Human	$.069\times10^{6}$ people		2.07×10^{6} l/dy	$.31\times10^{6}$	$.43\times10^{6}$	
	Industrial	14		5325.82×10^{6} l/yr	2.22×10^{6}	3.11×10^{6}	12.681×10^{6} tons/yr of iron ore
	Crops						
	Livestock	$.371\times10^{6}$ amu*		63.61×10^{6} l/dy	9.54×10^{6}	13.36×10^{6}	
M 5	Human	$.234\times10^{6}$ people		7.03×10^{6} l/dy	1.06×10^{6}	1.48×10^{6}	
	Industrial			0			
	Crops	10,000 ha		129×10^{9} l/yr	100×10^{6}	30×10^{6}	May through September
	Livestock	1.96×10^{6} amu		130.74×10^{6} l/dy	19.61×10^{6}	27.46×10^{6}	
M 6	Human	$.40\times10^{6}$ people		11.89×10^{6} l/dy	1.78×10^{6}	2.50×10^{6}	
	Industrial			0.0			
	Crops	10,000 ha		129×10^{9} l/yr	100×10^{6}	30×10^{6}	May through September
	Livestock	1.64×10^{6} amu		109.55×10^{6} l/dy	16.43×10^{6}	23.01×10^{6}	
M 7	Human	$.539\times10^{6}$ people		16.17×10^{6} l/dy	2.43×10^{6}	3.40×10^{6}	
	Industrial			0.0			
	Crops	200,000 ha		$6,220,000.00\times10^{6}$ l/dy	25.92×10^{6}	36.28×10^{6}	year round irrigation
	Livestock	$.223\times10^{6}$ amu		14.85×10^{6} l/dy	2.23×10^{6}	3.12×10^{6}	
M 8	Human	$.287\times10^{6}$ people		8.61×10^{6} l/dy	1.29×10^{6}	1.81×10^{6}	
	Industrial	16		7.38×10^{6} l/yr*	0.0	0.0	1.48×10^{6} KWH/yr of energy
	Crops	200,000 ha		$6,220,000.00\times10^{6}$ l/dy	25.92×10^{6}	36.28×10^{6}	year round irrigation
	Livestock	0.0		0.0			
M 9	Human	$.23\times10^{6}$ people		6.93×10^{6} l/dy	1.04×10^{6}	1.46×10^{6}	
	Industrial	16		7.38×10^{6} l/yr*	0.0	0.0	1.48×10^{6} KWH/yr of energy
	Crops	10,000		129×10^{9} l/yr	100×10^{6}	30×10^{6}	May through September
	Livestock	$.10\times10^{6}$ amu		6.67×10^{6} l/dy	1.00×10^{6}	1.40×10^{6}	

The energy inputs listed are produced by thermal power plants. Hydroelectric energy needs are reflected in low flow constraints. The demands are divided into 4 categories. The types in each category are listed in Table 2.5-7 with their water use coefficients.

2.5.9 Computer runs with least cost formulation of supply model: The six runs were done using the IBM (1971) Mathematical Programming System Extended (MPSX) Computer Package. The runs minimized total cost except for Run IV which minimized OMR costs. All runs have υ_i and μ_i in Equation (1) and (7) in Section 2.4.3 at .01 and, except the first, used drought hydrology.

Run I, Average Hydrology. This run used average values of rainfall and tributary inputs and maximum reservoir yields, as described in Section 2.5.7.

The initial projected total demands in M4 were too great for feasibility. Therefore, the livestock needs were decreased by 50 percent during each season. Thus, the livestock population was decreased from $.371 \times 10^6$ amu to $.186 \times 10^6$ amu. The water shortage occurred because the only sources of water in M4 are two unrecharged aquifers each with pumping limits of 10×10^6 m^3 annually. This change in demand was made for all subsequent runs also.

Run II, Drought Hydrology. This run used reduced values of rainfall, tributary flow, and reservoir yields as described in Section 2.5.7.

In the first run with the drought hydrology, an infeasible solution resulted because of low flow constraints in S1 during the dry season. To insure feasibility, the low flows during the dry season in M8 and S1 were decreased from 100 m^3/sec. and 3 m^3/sec. to 54.4 m^3/sec. and 1.1 m^3/sec. , respectively, for all drought runs. The reduced low flows in S1 and M8 will mean reduced river transportation,

Table 2.5-7

List of the Water Uses

	No.		Water Use
Human	1	Urban Population	30.0 l/day/capita
	2	Rural	30.0 l/day/capita
Industrial	3	Meat	200 l/ton/year
	4	Sugar	1800 l/ton/year
	5	Fish	16000 l/ton/year
	6	Rice	
	7	Milk	
	8	Beer	
	9	Wheat	
	10	Leather	
	11	Textile	
	12	Cement	550 l/ton/year
	13	Brick	
	14	Ore-iron	4200 l/ton/year
	15	Copper	3100 l/ton/year
	16	Energy	5 l/KWH
	17	Fertilizer	270,000 l/ton/year
Crops	18	Crop Land	1 l/ha./second
livestock	19	Cattle	66.7 l/amu/day
	20	Sheep and Goat	66.7 l/amu/day
	21	Horses	66.7 l/amu/day
	22	Pigs	66.7 l/amu/day
	23	Donkeys	66.7 l/amu/day
	24	Camels	66.7 l/amu/day

reduced power production, and reduced salinity control in these subareas.

Run III, Drought Hydrology with Minimization of Total Costs and Decreased Availability of Precipitation. In this run, the values of υ_i and μ_i in Equations (1) and (7) in Section 2.4.3 were reduced to 10 percent of their original values. This means that only .1 percent of the monthly rainfall that exceeds 15 mm per month is now available for direct use or catchment and storage or artificial recharge. Previously, 1 percent was available. The purpose of this run was to check the model's sensitivity to these parameters.

Run IV, Drought Hydrology with Minimization of the Operation, Maintenance, and Replacement, (OMR) Costs. The purpose of this run was to determine the effects upon project selection of minimizing the OMR costs instead of the total costs. From the point of view of donor and recipient nations, the minimization of OMR costs may be desirable because many projects in the Sahel-Sudan have failed because they have not been properly maintained.

Run V, Drought Hydrology with Reduced Crop Demands in M7 and M8. Preliminary analysis of the previous runs indicated that the large crop demands in M7 and M8 during the dry season probably necessitated the upstream storage in M7. Therefore, it was decided to investigate this by decreasing the crop demands in M7 and M8 in the dry season from $3,628 \times 10^6 m^3$ to $1,000 \times 10^6 m^3$. In other words, the fields would lay fallow for 4-5 months of the year.

Run VI, Drought Hydrology with New Demand Location Data in Subareas S4 and S5. This run reflects an updating of the solution set based upon hypothetical field work that found the piping distances in S4 and S5 should be reduced 50 percent and 30 percent respectively. This results in lower costs for surface water withdrawals because the transfer distance is less.

2.5.10 Results of computer runs. The results of the computer runs are shown in Tables 2.5-8 and 2.5-9. The figures in Table 2.5-8 show the quantities of water in 10^6 m^3 taken from each source in the wet season (first number) and dry season (second number). There are a few exceptions to this. The figures given for large and small scale storage are usually the amounts stored in each season. (These figures are only indicative of the use of these sources. Often, the runoff collected in a specially treated catchment during one season is used during that same season.)
In the subareas where water is taken from rivers, the operating policies and annual volume yields of the reservoirs are noted, as well as the seasonal volumes taken from the reservoirs and the low flow volumes at the exit of the Subareas (I_{ijt} in Equation 12 in Section 2.4.3). Shown in Table 2.5-9 are the capital, OMR, and total investment costs for the different runs. Table 2.5-10 shows the total investment cost breakdown for Run II.

General Character of Results

The solution sets appear realistic, given the large projected demands for 1990. The Continental Terminal formation (an excellent groundwater water source) is used whenever possible, and little use is made of the rock aquifer formations, which are poor sources. The large scale storage and use of rainwater occurs where it is presently utilized (i.e., south-central and southeastern Mauritania). Substantial use is made of the Senegal River for irrigation. (It is worth noting that this result corresponds to future OMVS plans for the river.) Finally, the suggested use of artificial recharge, recycling, and water importation in subarea M1 shows the critical water shortage in this subarea; these are certainly viable alternatives to the present desalination in this subarea, both in terms of expense and complexity. The reasonableness
(text continued on page 224)

Supply Sources
(Quantities in 10^6 m^3)
TABLE 2.5-8
Subarea Ml

Groundwater		Run 1	Run II	Run III	Run IV
Aquifer Number	Type				
2	Continental Terminal	0 18.41	3.97 6.03	24.36 0.0	3.97 6.03
3	Ephemeral	24.62 0	38.60 0.0	38.60 0.0	38.60 0.0
5	Continental Terminal (local)			21.92 31.15	
Surface Water					
Imports					
	From M8	7.94 11.29	17.78 25.26	10.20 14.50	18.52 26.31
Other					
Desalination					
Small Scale Storage of Rainfall and Runoff		2.0 0.0	2.0 (Caught and used in wet season)	0.0 0.0	2.0 (Caught and used in wet season)
Large Scale Storage of Rainfall and Runoff					
Recycling		.92 1.31	.92 1.31	.92 1.31	0.0
Artificial Recharge (1) to 2		8.41 0	0.0 0.0	0 14.36	0.0

(✓= possible source)

Supply Sources
(Quantities in $10^6 m^3$)

TABLE 2.5-8
Subarea M 2

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
2	Continental Terminal	0 .3	0 .3	0 .3	0 .3
3	Ephemeral	.20 0	.20 0	.20 0	.20 0
Surface Water					
Imports					
Other					
Desalination	✓				
Small Scale Storage of Rainfall and Runoff	✓	2.0 0.		2.0 0	2.0 0
Large Scale Storage of Rainfall and Runoff					
Recycling	✓				

(✓ = possible source)

Supply Sources
(Quantities in $10^6 m^3$)

TABLE 2.5-8
Subarea M3

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Rock				
2	Sand				
3	Ephemeral	1.35 0	1.35 0	1.35 0	1.35 0
4	Nubian Sandstone				
Surface Water					
Imports					
Other					
Desalination					
Small Scale ✓ Storage of Rainfall and Runoff					
Large Scale ✓ Storage of Rainfall and Runoff		8.53 0.0	8.53 0.0	8.53 0.0	8.53 0.0
Recycling ✓					

(✓= possible source)

Supply Sources
(Quantities in $10^6 m^3$)

TABLE 2.5-8
Subarea M 4

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Rock				
2	Continental Terminal	4.19 5.81	4.19 5.81	4.19 5.81	4.19 5.81
3	Ephemeral				
4	Nubian Sandstone	2.20 3.13	2.20 3.13	2.20 3.13	3.11 4.41
Surface Water					
Imports					
Other					
Desalination					
Small Scale Storage of Rainfall and Runoff	✓				
Large Scale Storage of Rainfall and Runoff	✓				
Recycling	✓	.91 1.29	.91 1.29	.91 1.29	

(✓ = possible source)

Supply Sources
(Quantities in $10^6 m^3$)

TABLE 2.5-8
Subarea M5

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Sand	0.0		37.17	
		0.0		52.83	
2	Rock	0.0			
		0.0			
3	Ephemeral	6.10	6.10	71.46	6.10
		0	0	0.0	0
Surface Water					
Imports					
Other					
Desalination					
Small Scale Storage of Rainfall and Runoff	✓	63.68	63.68	0.0	63.68
		0.0	0.0	0.0	0.0
Large Scale Storage of Rainfall and Runoff	✓	10.00	10.00	7.64	10.00
		0.00	0.00	0.0	0.0
Recycling	✓				

(✓ = possible source)

Supply Sources
(Quantities in $10^6 m^3$)
TABLE 2.5-8
Subarea M 6

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Rock			2.46 3.49	
2	Continental Terminal (local)				
3	Ephemeral	5.29 0	5.29 0	99.31 0	5.29 0
4	Sand	0 0.0	0 0.0	0 38.76	
Surface Water					
Imports					
Other					
Desalination					
Small Scale Storage of Rainfall and Runoff	✓	59.39 0.0	59.39 0	6.26 0	59.39 0
Large Scale Storage of Rainfall and Runoff	✓	10.00 0	10.00 0	10.00 0	10.00 0
Recycling	✓			.18 .25	

(✓ = possible source)

TABLE 2.5-8

Subarea M^7

Supply Sources
(Quantities in $10^6 m^3$)

Groundwater	Run I	Run II	Run III	Run IV	Run V	Run VI
Aquifer Number / Type						
1 Rock						
2 Maestrichtian						
3 Ephemeral	.67 0	.67 0	.67 0	.67 0		.67 0
Surface Water						
Senegal River, Withdrawal	2508.15 3558.25	2502.15 3634.52	2581.21 3634.52	2503.15 3632.52	2503.15 1004.52	2502.15 3634.52
Operating Policy of reservoir	(6242.93)* (8903.56)	(5959.)* (8406.74)	(6433.35)* (9029.24)	(5995.00)* (8414.63)	(13766.79)** 2204.69	6055.91 8499.53
Total Annual Yield	15233.65	14375.52	15440.00	14388.00	16973.50	14534.20
Low Flow at Exit	4188.72 5885.08	4211.33 5126.14	4252.64 5620.05	4212.07 5138.12	10423.05 2255.72	4211.33 5200.38
Exports to M7						
Exports to S3						
Exports to S4						
Other						
Desalination						
Small Scale Storage of Rainfall and Runoff	82.84 0.0					
Large Scale Storage of Rainfall and Runoff	50.0 (caught and used in dry season)	49.92 (caught and used in wet season)	10.39 (caught and used in wet season)	48.92 (caught and used in wet season)	50.00 (caught and used in wet season)	49.92 (caught and used in wet season)
Recycling						

* = Constant Yield
** = Natural

(✓ = possible source)

TABLE 2.5-8

Subarea M^8

Supply Sources

(Quantities in 10^6 m^3)

Groundwater		Run I	Run II	Run III	Run IV	Run V	Run VI
Aquifer Number	Type						
1	Alluvial	13.22 18.79	.45 .63	.45 .63	.45 3.56	2.11 3.00	.45 .63
2	Ephemeral						
Surface Water							
Senegal River, Withdrawal		2533.70 3578.86	2556.31 3654.44	2597.62 3643.68	2557.05 3652.57	2581.80 1002.34	2556.31 3654.44
Low Flow at Exit		1320.00 1840.00	1320.00 1174.19	1320.00 1576.84	1320.00 1185.24	6253.97 1000.00	1320.00 1233.4
Exports to M1		7.94 11.29	17.78 25.26	10.20 14.50	18.52 26.32	17.78 25.26	17.78 25.26
Exports to S3							
Exports to S6							
Other							
Desalination							
Small Scale Storage of Rainfall and Runoff	✓	54.31 0.0	27.16 (caught and used in wet season)		27.16 (caught and used in wet season)	27.16 0.	27.16 (caught and used in wet season)
Large Scale Storage of Rainfall and Runoff							
Recycling	✓						

(✓ = possible source)

Supply Sources
(Quantities in $10^6 m^3$)

TABLE 2.5-8
Subarea M9

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Rock				
2	Sand			75.34 32.86	
3	Ephemeral	.61 0	.61 0	.61 0	.61 0
Surface Water					
Imports					
From M7					
Other					
Desalination					
Small Scale Storage of Rainfall and Runoff	✓				
Large Scale Storage of Rainfall and Runoff	✓	41.08 0	41.08 0		41.08
Recycling	✓				

(✓ = possible source)

Supply Sources
(Quantities in $10^6 m^3$)
TABLE 2.5-8
Subarea S1

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
2	Continental Terminal	121.51 172.68	359.43 255.38	602.57 257.36	360.75 253.18
3	Ephemeral				

Surface Water	Run I	Run II	Run III	Run IV
Casamance River, Withdrawal	98.68 5.59	25.09 11.79	25.09 11.79	25.09 11.79
Low Flow at Exit	38.80	38.80	38.80	38.80
Imports	54.4	20.00	20.00	20.00
Other				
Desalination				
Small Scale ✓ Storage of Rainfall and Runoff	103.10 0.0	0.0 2.2	0.0 .22	Slight decreased use of storage
Large Scale Storage of Rainfall and Runoff				
Recycling ✓				

(✓= possible source)

Supply Sources
(Quantities in 10^6 m^3)

TABLE 2.5-8
Subarea S 2

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Continental	33.74	159.41	389.45	228.65
	Terminal	47.95	226.53	289.74	497.78
3	Ephemeral				
4	Maestrichtian				
Surface Water					
Export to S6		140.92	143.55	144.00	290.40
		200.25	203.99	204.63	412.67
Other					
Desalination					
Small Scale ✓		296.77	78.21	0.0	Approx-
Storage of Rainfall and Runoff		0.0	0.0	0.0	imately same amt. of storage
Large Scale Storage of Rainfall and Runoff					
Recycling ✓					

(✓ = possible source)

Supply Sources
(Quantities in 10^6 m^3)

TABLE 2.5-8

Subarea S3

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Continental	0.0	5.93	112.67	0
	Terminal	0.0	8.43	77.87	13.17
3	Ephemeral				
4	Maestrichtian				
Surface Water					
Imports					
From M8					
Exports to S6					
Other					
Desalination					
Small Scale Storage		✓97.34	86.80		80.87
of Rainfall and Runoff		0.0	0.0		0.0
Large Scale Storage of Rainfall and Runoff					
Recycling ✓					

(✓ = possible source)

TABLE 2.5-8
Subarea S4

Supply Sources
(Quantities in 10^6 m^3)

Groundwater		Run I	Run II	Run III	Run IV	Run VI
Aquifer Number	Type					
1	Rock					
2	Continental Terminal (local)				0.0	
					0.0	
3	Ephemeral	89.57	82.64	82.64	82.64	
		0.0	0	0	0.0	
Surface Water						
Faleme River, Withdrawal		104.11	284.88	433.63	284.88	367.52
		147.94	147.94	147.94	147.94	147.94
Low Flow at Exit		1005.81	1528.55	1215.71	1525.28	1462.44
Imports		2005.18	1634.61	1629.49	1640.29	1634.61
From M7						
Other						
Desalination						
Small Scale Storage of Rainfall and Runoff	✓					
Large Scale Storage of Rainfall and Runoff	✓					
Recycling	✓					

(✓ = possible source)

TABLE 2.5-8

Subarea S5
Supply Sources
(Quantities in 10^6 m^3)

Groundwater	Run I	Run II	Run III	Run IV	Run VI
Aquifer Number / Type					
1 Rock					
2 Continental Terminal (local)					
3 Ephemeral	265.46 14.18	132.83 7.10	132.83 7.10	132.83 7.10	132.83 7.10
Surface Water					
Faleme River Withdrawals	62.75 89.18	264.42 272.39	506.72 278.78	268.51 265.29	264.42 272.39
Operating Policy of reservoir	0.0* 2729.42	1750.25* 2456.49	1750.25* 2456.49	1750.25* 2456.49	1750.25* 2456.49
Total Annual Yield	4667.30	4200.60	4200.60	4200.60	4200.60
Low Flow at Exit	202.71 2654.42	1618.65 2191.20	1376.37 2184.81	1614.57 2198.30	1681.65 2191.20
Imports					
Other					
Desalination					
Small Scale ✓ Storage of Rainfall and Runoff					
Large Scale ✓ Storage of Rainfall and Runoff	192.66 0	0 7.1	0 .71	Approximately same use of specially treated catchments	0 7.1
Recycling ✓	.53 .75	.54 0	.54 0		.54 0

* = Constant Yield

(✓ = possible source)

Supply Sources
(Quantities in 10^6 m^3)

TABLE 2.5-8
Subarea S6

Groundwater		Run I	Run II	Run III	Run IV
Aquifer Number	Type				
1	Alluvial	6.70	5.90	6.54	5.90
		8.43	4.69	4.05	4.69
3	Ephemeral	1.21	.61	.61	.61
		0.0	0.0	0.0	0.0
Surface Water					
Imports					
From S2		140.92	143.55	144.00	290.40
		200.25	203.99	204.63	412.67
From S3					
From M8					
Other					
Desalination					
Small Scale ✓ Storage of Rainfall and Runoff		1.21 caught and used in wet season	.61 caught and used in wet season	.061 caught and used in wet season	1.21 caught and used in wet season
Large Scale Storage of Rainfall and Runoff					
Recycling ✓		146.84	146.84	146.84	0.0
		208.68	208.68	208.68	0.0

(✓ = possible source)

Table 2.5-9

Investment Costs

($\$ \times 10^6$)

Run	Capital Cost	OMR Cost	Total Cost
I	2753.38	531.25	3284.63
II	4068.75	664.48	4733.23
III	6121.02	844.13	6965.15
IV	4569.39	573.79	5143.18
V	3781.46	601.99	4383.45
VI	3394.76	607.83	4002.59

Table 2.5-10

Total Investment Cost Breakdown for Run II

(Drought Hydrology)

Source		Cost ($\$10^6$)	
Reservoir in M7		296.71	
Reservoir in S5		51.66	
Surface Water Withdrawal	M7	60.57 (OMR) 55.24 (C)	
" "	M8	61.30 (OMR) 55.55 (C)	
" "	S1	.36 (OMR) .54 (C)	
" "	S4	31.89 (OMR) 575.34 (C)	includes cost of transporting 42.5 km to center of subarea.
" "	S5	89.86 (OMR) 1312.23 (C)	includes cost of transporting 105 km to center of subarea.
Total Cost for Withdrawals		2242.83	
" Reservoirs		348.37	
Total Cost for Groundwater		518.93	
113 km pipeline from M8 to M 1		101.35	
125 km pipeline from S2 to S6		909.87	
40,000 m^3/hr. recycling plant in Dakar		604.35	
Total costs for all other sources		7.53	

OMR = Operating, Maintenance, Replacement Costs
C = Capital Costs

of the solution set is also evident in subarea S6, Dakar, even though in the solution set Dakar does not import water from the Senegal River (subarea M8) as it does presently. Instead of using this source of imported water, S6 relies upon water transferred from S2 (drawn from the Continental Terminal formation there) and recycling. S2 is the source of imported water instead of M8 because of less expensive transmission costs (less distance) and because it is more economical to use the Senegal River for irrigation and navigation than as a supply source for Dakar. So much recycling is possible in S6 because of the high industrial use.

Even though the model suggests using recycling for five of the subareas (indicating perhaps a too high level of complexity for developing nations), only in Dakar is the amount significant. Dakar can certainly provide the necessary maintenance support for recycling. For similar reasons, the significant amount of artificial recharge to the nonrecharged Continental Terminal formation in M1 is also feasible.

At a glance, the costs of developing and operating the water resources of Senegal and Mauritania may seem unreasonably high. However, the cost breakdown of the costs for drought conditions shown in Table 2.5-10 reveal their reasonableness. The reservoirs cost about $340 million to construct and operate. This figure compares well with quoted figures for the Manantali Dam in Mali. The total cost of the solution set would be significantly decreased if it did not cost at least $1500 million (approximately one-third of the total cost) to supply Dakar with water. Finally, the cost of supplying surface water would be decreased significantly if a 30 foot head differential had not been assumed.

<u>Comparison of Average Year and Drought Year</u>

Designing for the drought year increases the total cost by approximately 50 percent (from $3,300 million to $4,700 million). The effect of the drought is to decrease the precipitation caught for surface storage and to supply water from increased groundwater pumping and surface water withdrawals. The groundwater is not being mined more than in average hydrologic conditions. The reservoirs have more evenly distributed yields and lower total yield because of the reduced low flow constraints in M8 during the dry season. Given the present water demands in the Casamance region (S1), the drought is particularly severe there, resulting in significantly decreased low flows. However, the Continental Terminal aquifer supplies the water that is no longer available from the Casamance River.

Another interesting comparison is that generally, the sources of water do not change when the drought occurs. Therefore, if this were to hold for other runs, it would mean the unused sources have been "screened out" and only the used sources should be further studied.

In terms of cost, the solution set is sensitive to the design year. Therefore, several design years should be chosen and trade-offs developed between cost and the implied reliability of the water supply system.

Reduced Values of Readily Available Precipitation (Run III)

Compared to the drought year run (Run II), the solution set is sensitive to the values of the parameters ν_i, μ_i in Equations (1) and (7) in Section 2. 4. 3. Some new supply sources were introduced, and the total cost increased from \$4700 million to \$7000 million. Therefore, these parameters should be investigated; it is also possible that the values of these parameters could be greater.

Minimizing OMR Costs (Run IV)

Minimizing OMR costs does not change the nature of the solution significantly. The capital costs increased by approximately \$500 million (10 percent) and the OMR cost decreased by approximately \$90 million (14 percent). The major change was the increased capital intensive piping of water from S2 to Dakar to replace the use of recycling, which has a high OMR cost. This suggests that since a pipeline is probably more reliable than a recycling plant, Dakar should rely upon water pumped from S2 rather than a recycling plant. The solution set also points out that by designing to minimize total costs, operation and maintenance factors are still close to optimal.

Reduced Agricultural Demand in the Senegal Valley and Delta (Subareas M7 and M8) (Run V)

The most significant feature of this run is that an upstream reservoir is not needed in M7. This, and the decreased surface water pumping in M7 and M8 results in a cost savings of approximately $400 million.

Reduced Piping Distance in S4 and S5 (Run VI)

This run determines the effect of some hypothetical new information on the system. The piping distances in subareas S4 and S5 have been reduced 50 percent and 30 percent, respectively. The only significant changes in the solution are the cost savings of approximately $700 million and the replacement of the ephemeral aquifer in S4 by water from the Senegal River. Therefore, while the solution set as a whole is not effected by this information, subareas S4 and S5 are.

These computer runs are only a sample of the many least cost runs that can be done in an actual planning process. Additional possible runs are discussed in Section 2.5.13. Suitable planning processes are discussed in Part III.

2.5.11 Introduction: Computer runs maximizing net benefits These runs were made for two purposes: to determine the benefits that must correspond to the demands in Tables 2.5-5 and 2.5-6 in order to economically justify water resource development in the area; and to study water allocations under the assumption that no goods, including agricultural goods, are produced for export. These runs were done for subarea S1 (the Casamance), using drought hydrologic conditions (Run II in the least cost formulation).

To better model the benefits, piecewise linear benefit curves as shown in Figure 2.5-3 were used. Since we are maximizing net benefits, concave benefit curves can be used and global optima still result. Point D in the figure is called the first "breakpoint", point E the second "breakpoint", etc. The slope of $\overline{OA}$ is the unit value associated with all demands up to D, the slope of $\overline{AB}$ to all demands between D and E, etc. Upper bounds were put on the total amount demanded by any use. The bounds for human, industrial, crop, and livestock were 9×10^6 m^3, $.04 \times 10^6$ m^3, $1,000 \times 10^6$ m^3, and 10×10^6 m^3 respectively.

Determination of Approximate U.S. and Senegal Benefits To provide a basis for comparison, the benefits in the United States from these four water uses were obtained. It was assumed that what users presently pay for water approximate these benefits, and these benefit approximations are net of willingness to pay for distribution costs. Once the unit payments for these uses were obtained, the present values of the payments for 50 years of expenditure and discount rate of ten percent were calculated. The results are in Table 2.5-11. The sources of U.S. water costs were Hirshleifer et al. (1960) and Bain et al. (1966). For illustrative purposes only we assumed that these benefits approximate those of users whose products are sold on the "world market". In order to approximate the benefits of water uses which only produce products for the "home

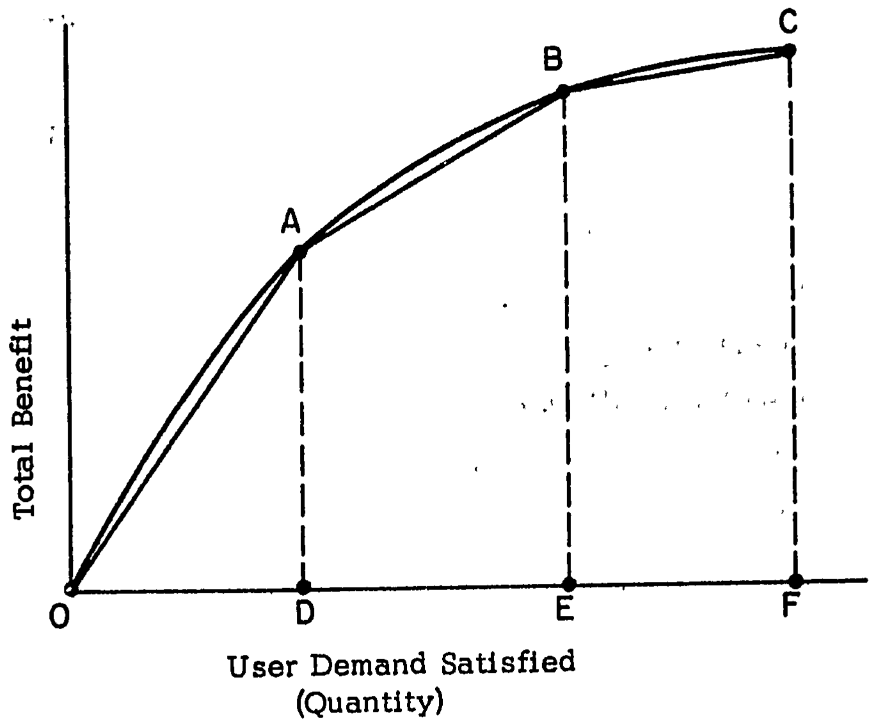

FIGURE 2.5-3 Piecewise Linear Benefit Curve

TABLE 2.5-11

Discounted Benefits of Water Use in United States and Senegal

Water Use	Benefit ($\$/m^3$) U.S.	Senegal
Human	2.6	.12
Industrial	1.74	.08
Crop	.20	.01
Livestock	.20	.01

market" in Senegal (i.e., developing self-sufficiency in Senegal), the U.S. benefits were multiplied by the ratio of Senegal per capita income to U.S. per capita income. The sources of the per capita incomes were U.N. (1972).

Runs and Results: The results are shown in Table 2.5-12, along with the result from the least cost run. (Run II is least cost formulation)

Run 1 determined the benefits of the water uses such that approximately the same demands occurred as in the least cost run. These benefits were determined in an iterative procedure, trying different sets of benefits until a satisfactory set was found. The starting points were determined by calculating the unit costs of supplying the water. As can be seen in the results, the industrial benefits lie between the "world" and "local market" benefits in Table 2.5-11. The agricultural benefits (crops and livestock) equal those of the world market benefits. Therefore, a large investment in water supply in this subarea is only justified if the agricultural products are to be sold on the "world market" (given our assumptions). Calculating the user benefits such that the benefits just exceed the costs, given these demands, results in benefits of $\$.30 \times 10^6/m^3$ for human use, $\$.20 \times 10^6 m^3$ for industrial use, $\$.12 \times 10^6/m^3$ for agricultural use. Therefore, even if the benefits were to be decreased by $\$85,00 \times 10^6$, the large scale irrigation could still not be justified by production for "local markets" only.

Run II was a series of runs to determine the optimal uses of water given the "world market" benefits and budget constraints of $10.00 to $50.00 million. This illustrates the situation when a donor agency is deciding how to allocate its financial resources, given budget constraints. The net benefits that result from each budget constraint are shown in Table 2.5-13.

TABLE 2.5-12

Results of Net Benefit Runs

	Least Cost	Run I	Run II*	Run II**	Run III & IV
Annual Groundwater Use	614.81	445.15	62.56	341.47	0.0
Annual River Water Use	36.88	36.88	36.88	36.88	0.0
Volume of Storage Needed	137.74	135.74	134.42	134.42	134.42
Annual Human Use	8.53	9.00	9.0	9.0	9.0
Benefits(and Breakpoints)	______	.30(∞)	same	same	.12(∞)
Annual Industrial Use	.03	.04	.04	.04	.04
Benefits(and Breakpoints)	______	.21(∞)	same	same	.08(∞)
Annual Crop Use	908.00	735.35	353.64	632.79	241.49
Benefits(and Breakpoints)	______	.2(700), .05(∞)	same	same	.01(50), .005(∞)
Annual Livestock Use	7.49	10.00	10.0	10.0	10.0
Benefits(and Breakpoints)	______	.18(∞)	same	same	.01(25), .005(∞)
Total Cost	114.38	64.82	10.00	50.00	.14
Net Benefits	______	85.80	65.54	81.27	2.75

Units of Costs and Benefits are $\$/m^3$

Units of Demands and Supplies are 10^6 m^3.

*Budget Constraint equals $10.00 million
**Budget Constraint equals $50.00 million

TABLE 2.5-13

Net Benefits Given Budget Constraints

Budget Constraint $\$10^6$	Net Benefits $\$10^6$
10.00	65.54
20.00	69.47
30.00	73.41
40.00	77.34
50.00	81.27

The difference in national income benefits for each additional $10 million invested is approximately $3.94 million. Therefore, in this case, there are no large increases or decreases in the benefits as more money is invested. The reason is that at these levels of investment the additional water is always coming from the Continental Terminal aquifer and the incremental amount of water supplied are always the same.

Run III determined how water should be used if all the uses were for making products for local consumption. As can be seen in Table 2-5-12, the major change is in the water allocated to crops. 241.49×10^6 m^3 of water annually for crops corresponds to about 13,000 hectares of crops cultivated April through October. All the water comes from rainfall and storage of rainfall. This storage could take place in the swamps bordering the Casamance River, as is presently being proposed. The decision to undertake this investment would depend upon the priorities of decision makers and whether or not 13,000 hectares was enough cultivated area for self-sufficiency.

Run IV was to determine how to allocate the water uses if local labor and techniques were partly used to supply the water. To model this, the OMR costs of ephemeral groundwater and river water were reduced 75 percent, and the capital costs of ephemeral groundwater and continental terminal groundwater were reduced by 50 percent. The solution set did not differ from Run III. Another run that could have been done was to reduce the cost of river water use even more, to correspond to controlled flooding of crops. The efficiency of this source for irrigation also would have to be decreased to model this method's inefficiency.

2.5.12 Summary of results for both types of runs: In terms of a planning process the runs indicate the following:

1. Generally, the sources used in each subarea remain the same in each run. Therefore, unless a significant change occurs in a subarea as a result of a run (such as the use of the Continental Terminal formation in M1, and the Sand aquifer in M5 during Run III, or the switch from recycling to more transferred water in Dakar as a result of minimizing OMR costs), the sources in the solution are those that should be scheduled for study and development.

During drought periods, the solution indicates that increased use should be made of surface water and groundwater, instead of more recycling and desalination.

3. The cost of development is relatively sensitive to the design conditions. Therefore, the choice and implication of the design year must be considered very carefully. The trade-offs between system reliability and cost must be made explicit.

4. The solution set is sensitive to the amount of precipitation assumed readily available. Therefore, the values of the controlling parameters (ν_i, μ_i) should be checked by field work and other means.

5. Unless major changes are needed in the groundwater assumptions, there is presently physically adequate groundwater, even in times of drought. Very few of the groundwater constraints in the supply model are binding. Therefore, studies are needed on how to improve its management.

6. If Dakar reaches its projected demands, it will be very expensive (perhaps impossible) to provide it with adequate amounts of water. Perhaps a major effort should be undertaken to control the future water use in Dakar by limiting its population and some forms of industrial growth. An alternative to this is finding new water sources

that are close and inexpensive, or else trying to find less expensive ways of using the present known sources through new technologies.

It appears that transferring water from S2 is preferred to recycling to meet the projected demands on grounds of reliability.

7. Surface water as opposed to groundwater should be used to meet large scale agricultural needs in Senegal Valley and Delta. However, the cropping patterns must be analyzed. Cropping mainly in the wet season (perhaps April or May through September, October or November) may result in significant savings because upstream regulation would not be required. To provide water as needed, pumping could be used (as opposed to controlled flooding, which depends upon river depth). However, since pumping costs are high, the possibility of controlled flooding or other alternatives should be explored.

The trade offs in this area between maintaining the flow of the Senegal River during the dry season during droughts for navigation, power production, and pollution control, and withdrawing the water for irrigation and other uses, has to also be examined.

8. The demands in S1 are probably too high, given the flows of the Casamance River. Therefore, the demand values and/or the hydrologic data on the Casamance River should be reassessed.

9. If the piping distances in S4 and S5 are actually less than originally thought, significant savings will result in those subareas. However, the rest of the solution set remains essentially the same.

10. Given the data used, the results indicate that in the Casamance, large scale investment in water resources such as indicated in the least cost runs is only economically justified if the products can receive "world market" prices. However, it is still economically justified to produce fewer goods only for local market prices.

This requires a significantly smaller amount of investment. The decision between the two alternatives depends upon donor and recipient priorities.

2.5.13 Other important model uses: If the supply model were to be fully incorporated into a long term water resources planning process, the following types of additional runs should also be done:

1. Varying demands (both quantity and time) to determine costs of achieving different social objectives.

2. Incorporating more social and political constraints, to see how these constrain the solution. For example, requiring that the money invested in each subarea of a country be approximately the same or that wells must be at least one mile apart in some subareas to prevent overgrazing.

3. Varying more of the hydrologic coefficients to determine the solution set sensitivity to them as was done in Run III. Groundwater recharge assumptions should especially be checked.

4. Using more design years or conditions to determine tradeoffs between cost and reliability.

5. Studying the possible advantage of small scale, local development. This could be done by adjusting the cost coefficients of the sources to reflect the use of local materials and labor. Demand amounts could also be lowered.

6. Studying the possible advantage of new technologies by adjusting cost of coefficients to reflect this.

7. Incorporating proposed projects into the model to study how development should proceed given these projects. One of the several ways this could be done is to only consider the OMR cost of the source corresponding to the project if the project capacity is not exceeded. If the project capacity is exceeded, then an additional capital cost would have to be considered. This could be done by using

separable programming techniques.

8. Varying cost coefficients to determine the solution set sensitivity to them.

9. Varying low flow constraints to determine their effect on the solution set.

10. Running the model to show the physical and cost implications of the transition from traditionally based economy to a more modern economy.

2.6 Some perspectives on the supply model

2.6.1 Introduction: The purpose of this section is to evaluate the supply model in terms of its objectives and its major assumptions.

2.6.2 Supply model objective: As discussed in Section 2.4.1, the objection of the supply model is to indicate broad water resource investment strategies for the Sahel-Sudan region and to provide a framework for the analysis of water resource issues. We believe that the case study has illustrated that the supply model can satisfactorily achieve these objectives. The model has provided an analytical means for indicating specific water resource investments in the subareas of the case study as well as for determining broad water resource management strategies for the entire region studied in the case study.

2.6.3 Supply model assumptions: The major assumptions of the supply model as used for the runs described above are:

1. Temporal effects can be aggregated into two seasons.

2. Spatial effects can be aggregated. This assumption lets us assume that most demands are located at one place and each source is located at a particular place.

3. Costs are linear (discussed later on in this section).

4. The model is deterministic, and therefore relies upon sensitivity analysis to study the effects of uncertainties.

5. The model assumes all projects are built and put into operation simultaneously and operated for a period of fifty years, and that benefits and costs start flowing immediately upon implementation. This amounts to a static analysis of dynamic budgeting and scheduling effects which might not always be appropriate in planning.

6. Short run losses can be ignored. The model only plans for target demands. Obviously, as hydrologic and demand conditions change, the demands will not be satisfied all the time and short run losses will occur.

7. Local aquifer effects can be approximated by assuming constant well depths, lift heights, and yields for the various formations.

8. Overyear storage capacilities of recharged aquifers can be handled implicitly. Some aquifers have a potential to provide over-year storage as do reservoirs. The model does not explicitly consider this. However, overyear storage effects can be modelled by increasing the "mining coefficient", A_{ik}, in Equation (13b) in Section 2.4.3, or else fixing the amount recharged at 3a value equal to the average annual recharge when considering drought conditions.

9. The three simple reservoir operating procedures are adequate to characterize the possibilities of over-year storage.

10. Water quality issues can be modelled implicitly. The major water quality issues in the region are the provision of safe drinking water to inhabitants, salt build up in irrigated areas, and providing sufficient low flows in the river so that local stream pollution does not become a problem. If the quality of a source is doubtful for human consumption, the cost of utilizing the source can be increased to account for necessary treatment. Preventing salt

build up in irrigated areas can be modelled by increasing agricultural water needs to account for leaching water. Low flows are modelled by setting minimum streamflow values. The setting of the previous water costs and requirements demands judgement on, for example, what the salt build up will be, how serious the local pollution problems are, etc.

The assumptions are made for one or more of the following reasons:

1. The broad planning objectives for which the model was developed.
2. The nature and quality of data available.
3. Cost and ease of model operation.
4. Choice of approach to theoretical issues.

Assumptions related to the broad model objectives include the aggregation of temporal and spatial effects, the explicit neglect of uncertainties , the neglect of short run losses, the neglect of local aquifer effects, implicit modelling of overyear aquifer storage, and implicit consideration of water quality issues.

Assumptions related to data include the aggregation of spatial effects, the explicit neglect of all uncertainties except hydrologic, the use of static analysis, and the neglect of short run losses. Except for a few subareas, data were not available on the possible detailed spatial conditions of demands. Therefore, the model at a smaller scale would have been misleading. The same is true for measures of demand, social, technological, and political uncertainties. Static analysis is perhaps partly justified because data are not available on how benefits and costs change over time in the regions, and what the funding budgets are. Neglect of short run losses is justified because there are no data available for these losses in the region.

Assumptions made so that modelling would be economical, include the aggregation of temporal and spatial effects, the neglect of uncertainties, the use of static analysis, and the consideration of only three reservoir operation policies. If these three assumptions had not been used, the supply model would have contained so many integer variables and rows and continuous variables, that the cost of a single solution would have been prohibitively large (perhaps $2000). The present supply model costs about $20 per run. The additional information gained would not have been worth this price because of data uncertainties and the model objectives.

Assumptions reflecting approaches to theoretical issues include the explicit neglect of hydrologic uncertainty (and perhaps the other uncertainties), and the use of three simple reservoir operating policies. Hydrologic uncertainty must be explicitly neglected because of the theoretical difficulties with the two possible ways of incorporating uncertainty into linear programming models. The two methods are linear programming under uncertainty (LPUU) and chance constrained programming (CCP). As described by McBean (1973), LPUU requires too large an increase in problem size to be useful, must neglect serial correlations in successive hydrologic events when capacity decisions are being made, and requires the determination of penalty functions.

The major theoretical difficulty with CCP is the neglect of the correlations between hydrologic effects. The use of CCP in our case is even made more complicated because the supply model utilizes both stream flow and precipitation as direct supply sources. Therefore, the joint probabilities of stream flow and precipitation events must be studied.

The use of only three reservoir operating policies avoids both several theoretical difficulties and a large increase in the size of the supply model. A major problem with one year (even multi-seasonal) screening models that include reservoir storage is the incorporation of overyear storage effects. Obviously, by studying only one year, storage cannot be modelled explicitly. A common procedure in three one year models is to design the reservoir capacity based upon mean seasonal or monthly flows with the initial storage in the first period equal to the final storage in the last period. This is the method used by Cohon et al. (1973).

This procedure usually leads to "approximately correct" reservoir capacities that must be improved upon by simulation analysis using perhaps 00 years of synthetic data. This screening procedure also neglects consideration of what are physically feasible reservoir operating policies. A possible operating policy must be assumed for the simulation studies. An alternative to this screening approach is to model several or many years in the screening model. However, this method, while it may lead to better reservoir capacity estimates, also has several difficulties. It increases the problem size (Fergusson and Loucks(1972) suggest a method to condense the problem size), the reservoir capacities are sensitive to the hydrologic input traces used (Facet 1974), and the model still does not consider physically feasible operating policies. Both of these screening models also do not assign any sort of reliability measure to the yields of reservoirs unless probabilistic techniques are used. However, as discussed previously, these probabilistic techniques have their own inherent difficulties.

It is for these reasons it was decided to select reservoir capacities based upon fixed percentage yields in the different seasons.

Operating policies for simulation are more easily adapted to produce constant yields in each season than to produce the sequence of releases dictated by the two previously discussed screening models. In addition, if the yields from the reservoirs and their associated capacities are determined by either simple mass curve analysis or else by sophisticated simulated storage-yield curves, overyear effects can be easily incorporated by prior analysis. (However, mass curve analysis still has the problem that the yield and the capacity depend upon the hydrologic trace used). Both mass curve and storage-yield methods also assign a measure of reliability to the reservoir yield (such a measure might be the probability of the reservoir not meeting the demand in one season). Only an approximate measure of reliability can be determined from mass curve analysis (Fergusson and Loucks 1972) whereas simulated storage-yield curves produce a more accurate measure. Therefore, the three simple operating policies were chosen because, (1) it is possible to physically operate reservoirs to obtain their sequence of releases, (2) overyear storage effects are more easily incorporated, (3) a measure of reliability can be assigned the yields. Thus, this procedure avoids several theoretical (and model size) problems.

The alternative to assuming all unit costs are linear is to use piecewise linear approximations as in Figure 2.5-4a. However, since most curves are concave because of economies of scale, global optima do not always result when minimizing costs. Therefore, the procedure recommended is to assume the costs are linear, but choose the unit cost to correspond to the expected size of the project. If, after problem solving, the size is not as expected, the unit cost can be adjusted and the problem re-solved. For example, in Figure 2.5-4b, if the expected size before the solution was found was B, but after the solution was found the size was A, the unit cost at A should be used instead of the

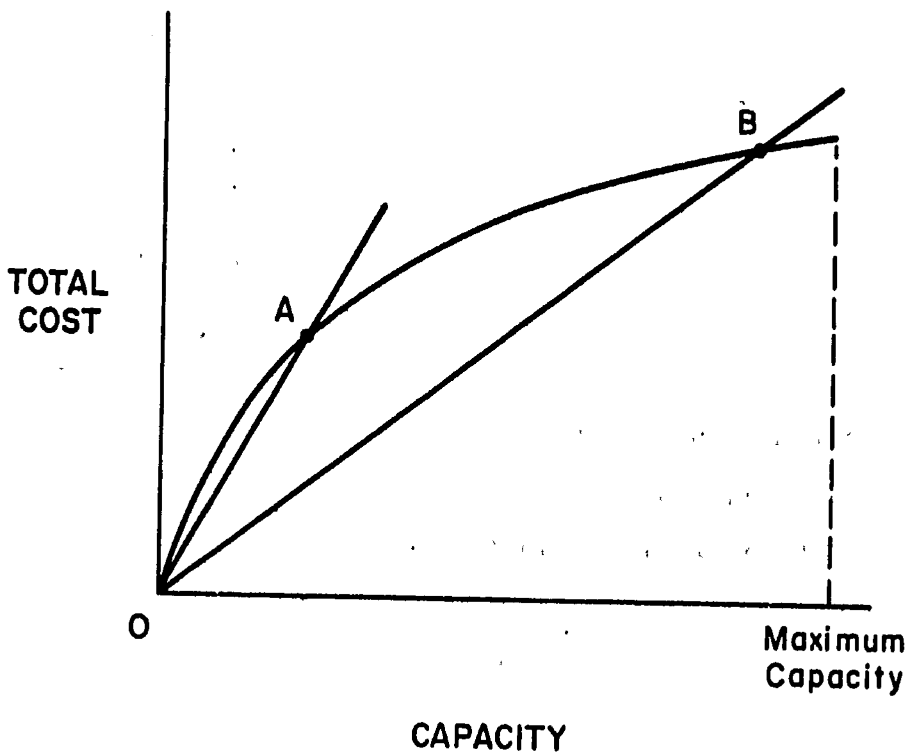

FIGURE 2.5-4b Determination of Correct Unit Cost

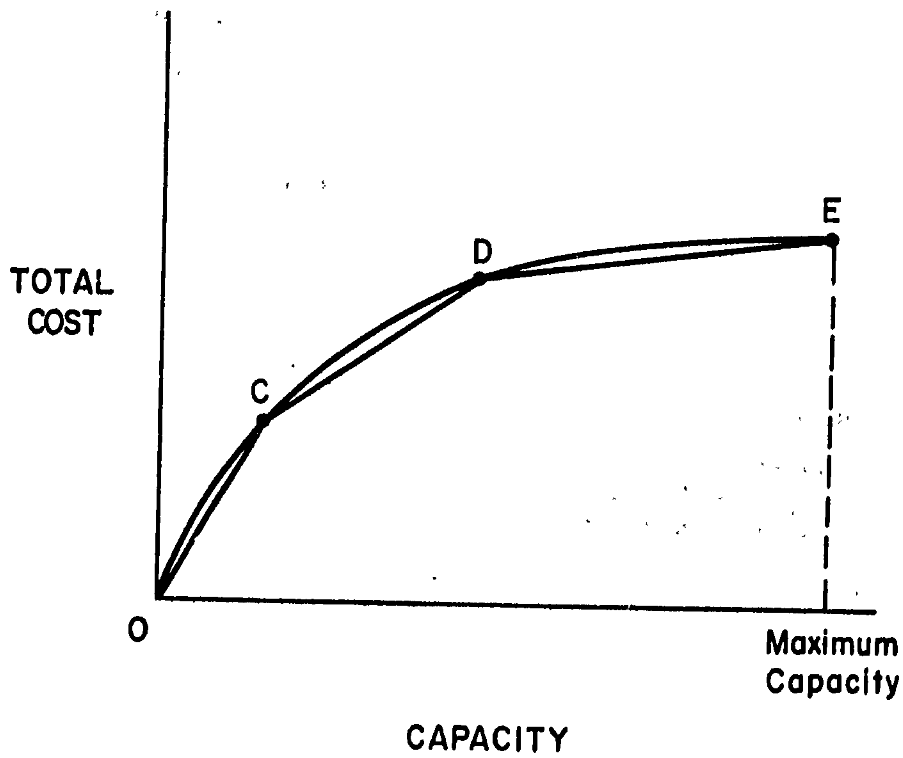

FIGURE 2.5-4a Piecewise Linear Curve Approximation

unit cost at B, and the problem re-solved until there is relatively good agreement. In terms of the least cost case study, only the large structures such as reservoirs, desalination plants, and recycling plants show significant economies of scale. However, even if piecewise linear approximations were used, the results would not be significantly different. The unit costs chosen for the recycling plant in Dakar and the reservoirs in S5 and M7 correspond to the capacities in the solution. Desalination would only probably enter the solution set if due to economies of scale, a very low unit cost was achieved. However, it would still not be a major source, and hence the low unit cost would not be justified. Therefore, desalination will probably never enter the solution even if cost nonlinearities are considered. Economies of scale are probably insignificant for small structures such as wells, small surface storage dams, etc. This is not only because they are small, but also because those unit costs are really the cost for constructing a number of small projects throughout an area. In other words, these small supply projects are not all built in one place and by one contractor. Rather, they are built throughout a subarea by different builders independently and economies of scale are not really possible. To summarize, assuming costs are linear is not a weak assumption if appropriate procedures are followed.

With further experience with the supply model and the region's hydrology, it may be useful to improve upon the supply model. However, presently, the authors believe the model is an appropriate initial planning tool for the planning framework developed here.

Possible changes that may be made in the supply model include:

1. Where it seems justified on the basis of analysis of the results of the supply model as now formulated, shorter time intervals might be used. If shorter time intervals were used (perhaps months instead of two seasons), the system capacities might be more accurately

calculated because hydrologic and demand peaks would be more sharply defined. The effects of rainfall and streamflow variability upon the satisfaction of demands would also be more explicitly studied if shorter time steps were used. In the two season model, more accurate system capacities could be calculated by multiplying the conversion factors in Equation set (16) in Section 2.4.3 by peak to average flow ratios to approximate the effects of peak demands.

2. Using smaller subareas. Smaller subareas would allow better representation than presently, of the uneven distribution of water sources and users. As a result, cost function may be more accurate as well as supply capacities.

3. Approximating nonlinear cost curves with piecewise linear functions. The problem with using piecewise linear cost functions is dealing with local optima. However, if the techniques suggested in this section are not adequate, piecewise linear approximation might be appropriate if results are checked for local optima.

4. Modelling reservoir storage with a different technique. Reservoir storage could have been modelled using the more traditional approach of:

Inflow and previous storage = outflow and present storage

This has several advantages because the operating policy is not fixed as it is now. However, this approach may require operating a monthly model for several or many consecutive years to obtain an accurate representation of the storage capacity needed.

5. Modelling aquifers in more detail so that overyear storage can be explicitly modelled. This may be particularly relevant where the present solution set suggests that large scale groundwater development is required, since such developments are likely to be used for over-year storage. (This aspect of storage of streamflow in reservoirs is dealt with in the present model through use of the storage-yield concept.)

PART III: A PLANNING PROCESS FOR LONG RANGE PLANNING OF WATER RESOURCES IN THE SAHEL-SUDAN REGION

III. A Planning Process for Long Range Planning of Water Resources in the Sahel-Sudan Region.

3.1 Introduction

The purpose of this part of the report is to discuss what we believe to be an appropriate planning process for the region, and how the supply and demand models can be used as central elements in this process. This discussion begins in Section 3.4. As a preliminary to this, we present some additional considerations on design hydrologic conditions and on benefits.

3.2 Determination of Design Hydrologic Conditions

3.2.1 Introduction: For any run of the supply model, three sets of hydrologic design values must be inputed. The sets are rainfall values, unregulated stream flow values, and the possible yields from reservoirs depending upon their operating policies. Ideally, some measure of the stochasticity of these values should be determined and related to the reliability of the resulting supply system suggested by the supply model.

3.2.2 Precipitation and unregulated stream flow: A standard procedure for the design of structures and projects that depend upon precipitation and unregulated stream flow is the design for an extreme event. For example, in the North Atlantic Regional (NAR) study of water supply for Northeastern United States (DeLucia and Rogers 1972), 7 day - 50 year low stream flows were used. The probability of the flows being less than these values was .02. Therefore, the probability of failure of a system depending directly upon these flows was .02. The 7 day - 50 year low flows were determined from a probabilistic analysis of the stream flow records.

Initially, a similar procedure was going to be recommended for use in selecting the hydrologic design conditions for the supply model. For example, the fifty year wet season and dry season low stream flows and rainfall values would be used. However, this approach has three theoretical difficulties. The first is that the fifty year (or any year) value of a parameter in the wet season does not necessarily occur simultaneously with the fifty year value in the dry season. In other words, a wet season of a particular return frequency is not necessarily followed by a dry season of the same return frequency. Therefore, to use fifty year values for each season would be using conditions that might never occur. The second problem is similar. A stream flow of a particular return frequency does not necessarily occur when the rainfall of the same return frequency occurs. Therefore, the use of 50 year (or any year) values of each correspond to conditions that might not occur. The third problem is that the stream flow or precipitation of a particular return frequency in a certain location does not necessarily occur when the stream flow or precipitation of the same return frequency occurs in a different location. The cause of this is the transformation process of rainfall to runoff and the north-south weather transition zones in the region. Therefore, a good design procedure appears to be to use the rainfall and stream flow values that are recorded in an actual year. This guarantees that the design is for hydrologic conditions that can occur simultaneously. Possible design years for the region studied in the case study are 1940 for the drought year and 1962 for an average year. The average annual flow of the Senegal River at Bakel in 1940 was 429 m^3/sec, 44 percent less than the average of 77 m^3/sec. The 1962 flow was approximately average (767 m^3/sec) (Senegal-Consult 1970). It has been said, the advantage of this approach is that events that can in fact occur simultaneously are studied. On the other hand, a possible problem is that no explicit measure of overall system reliability is determined.

An approximate measure could be gotten by either determining the return frequency of the precipitation of the design year at a central location or else determining the individual return frequencies of the design stream flows and precipitation values used in each subarea. It may well turn out that the recurrence intervals of the rainfall and/or the stream flows in the different subareas of the region or a subregion are approximately the same because their climate is determined by the same natural causes.

A possible difficulty in the model input data preparation is determining the area precipitation value to use. The value chosen must account for the spatial variation of rainfall in the subarea. There are several approaches. One is to weight the precipitation at each station in the subarea with the area the station is assumed to represent. This can be done using either a Thiessen network or an isohyetal map (Linsley and Franzini 1972). Another procedure is that reported by Rodriguez-Iturbe and Mejia (1974). The basis of this method is to determine the "reduction factor" for a subarea that converts point rainfall to areal rainfall. The reduction factor is determined from the "characteristic distance" of the subarea and the correlation between point rainfall at this characteristic distance. This procedure could either be applied to the entire subarea (i.e., use the point rainfall at one station in the subarea to determine the areal rainfall over the entire subarea) or else to several divisions of the subarea. In this latter method, the areal rainfall in each division would be determined using the method and then the results combined by either area weights or else weights based upon the years of record at each station.

3.2.3 Reservoir yield: Since reservoirs provide overyear storage (i.e., they can store stream flows for several consecutive years for release in later dry years), single year design procedures cannot be used. There are two basic methods used to determine the yields of regulated streams as a function of reservoir capacity. The first is mass curve analysis (Linsley and Franzini 1972). A mass curve (or Ripp diagram) is a graphical plot of cumulative stream flow into site versus time. Such a curve is shown in Figure 3.2-1. To determine the maximum constant yield of a regulated stream that can be obtained with a reservoir of a fixed capacity, "tangents are drawn to the high points of the mass curve in such a manner that their maximum departure from the mass curve does not exceed the specified reservoir capacity" (Linsley and Franzini 1972, p. 170.) The slope of steepest tangent is the maximum yield. Adjustments have to be made for evaporation and seepage losses. Similar procedures can be used to determine the yields for varying demands. Fergusson and Loucks (1972) estimate that if the number of years of record is n, the yield determined in this manner has a probability of $n/_{n+1}$ of being exceeded (i.e., the probability of shortage is $1-(n/n+1)$). If low flows are excluded from the stream flow record, the maximum yield of course increases. However, the chance of failure increases. Fergusson and Loucks (1972) estimate that if the m lowest flow years are excluded, the probability of exceedance of this yield is $\frac{n-m}{n+1}$. These estimates and mass curve analysis assume that the hydrologic traces analyzed always repeat themselves exactly.

The other method to determine storage capacities necessary to achieve certain yields is by using computer simulation of stream flows, reservoir operation, and demands (Maass et al. 1962). The major advantages of this method are that significant improvements can be made in the estimates of failure to meet demand and more flexibility is possible in the analysis concerning operating policies, hydrologic traces studied, etc.

The choice of methods really depends upon the tools available to the study team. However, in determining the storage-yield curves,

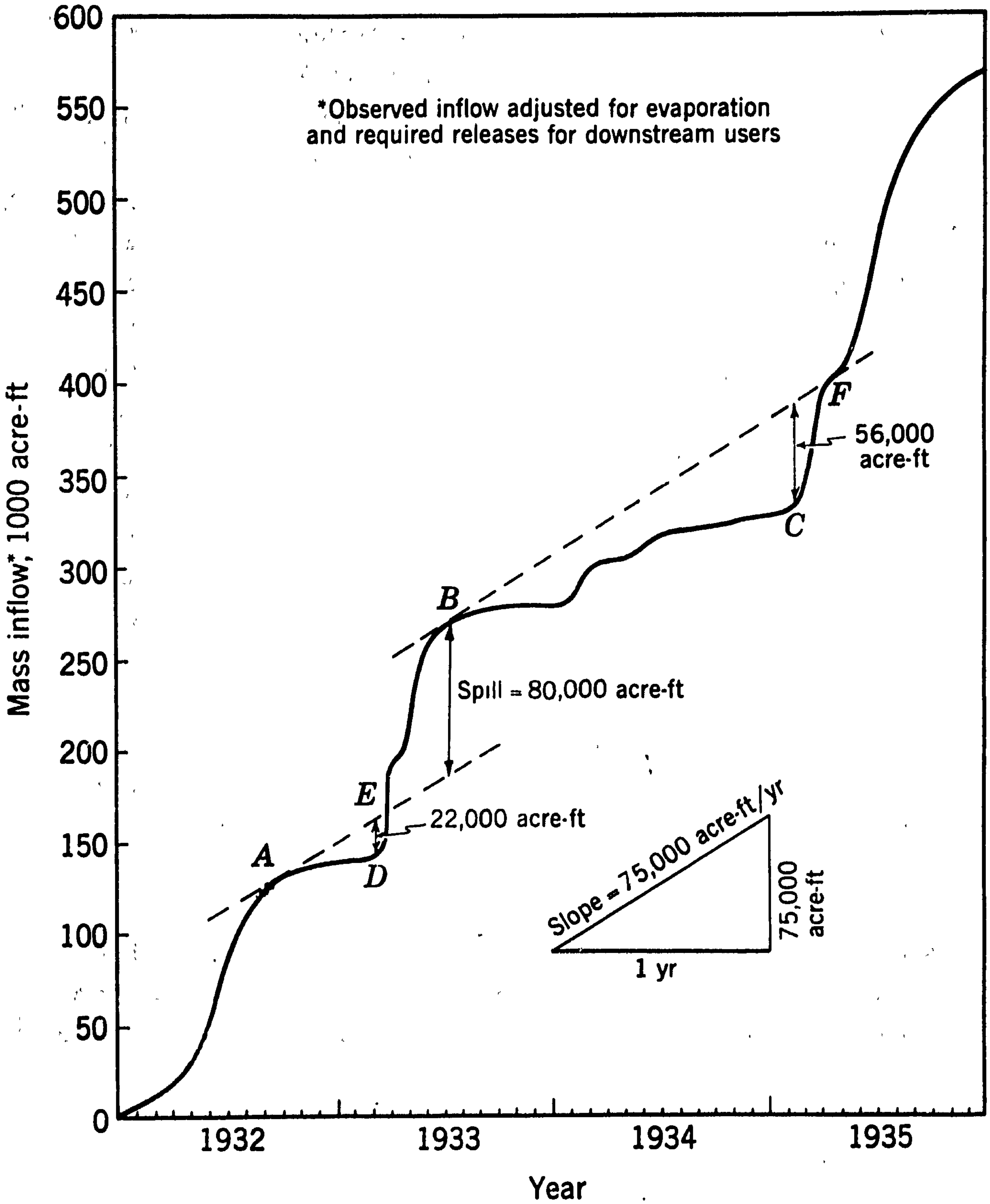

FIGURE 3.2-1 Mass Curve

Source: Linsley and Franzini 1972. Courtesy of McGraw-Hill Publishing Company.

failures should not be allowed to occur during the design years chosen for the selection of the other hydrologic inputs. Otherwise, the yield assumed available during that period will not really be available because that is when the reservoir has failed to meet demands.

3.3 Determination of Design Demands or User's Benefits

3.3.1 Design demands:

The purpose of the demand values used in the cost minimization formulation of the supply model is to represent the social welfare benefits of development as they relate to water resource development. In other words, if the demands used in the supply model can be satisfied, it should imply a certain level of social welfare can be reached. Tradeoffs can then be made between the achievement of various levels of social welfare and cost. Therefore, the demand values should not be arbitrarily chosen. They should be chosen as the water demands that correspond to possible desirable patterns of development. For example, if decision makers decide that in certain areas, irrigated agriculture will significantly improve the welfare of the area's inhabitants, the different amounts of water that are required for different levels of irrigation should correspond to different levels of social welfare for the area's inhabitants. The model would determine the different cost of supplying the water and theoretically, a curve could be drawn of cost versus social welfare, as in Figure 3.3-1.

The demand model discussed in Section 2.3 is useful for the task of translating production goals into water needs.

3.3.2 Determination of benefits

The ideal use of the supply model would be to use it to select the projects and uses that maximize net social benefits. Social benefits have sometimes been defined in economic terms alone, but certainly,

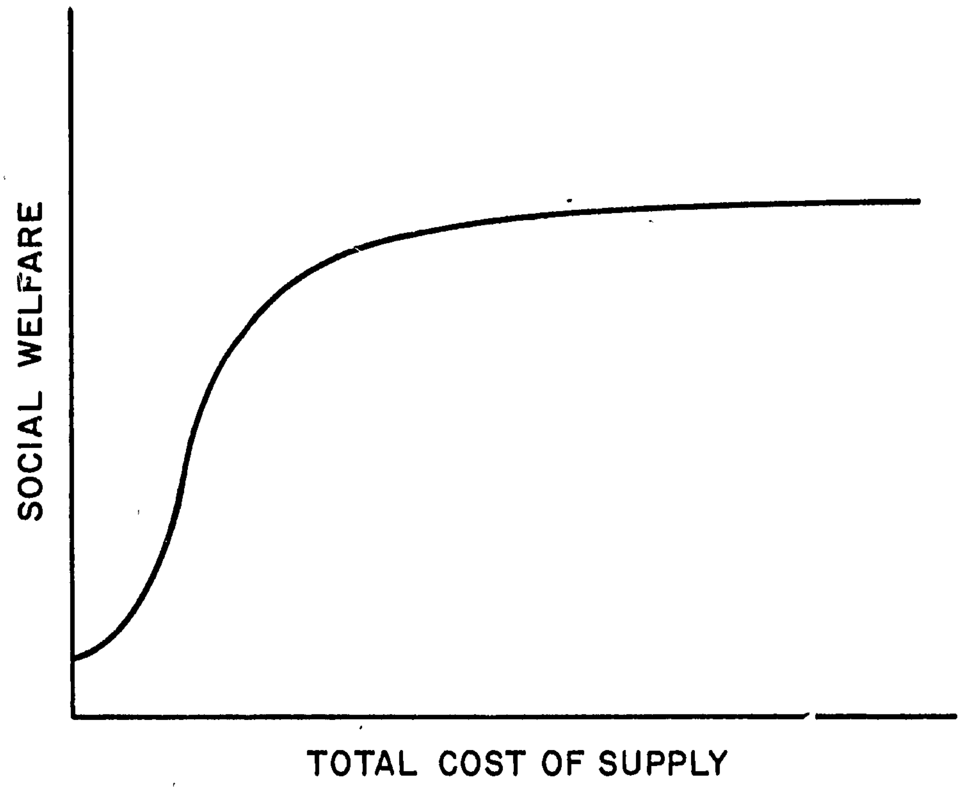

FIGURE 3.3-1 Social Welfare Versus Cost

in the Sahel-Sudan region social benefits reflect multiple objectives of investment and management schemes, emphasizing regional, group, and health objectives as well as purely economic objectives. Fortunately, a methodology for considering such multiobjective problems has been developed in the water sector, as for example in Maass (1962), Marglin (1967), and UNIDO (1972). This approach involves the utilization of appropriate metrics representing social objectives in the design of projects and programs. It is appropriate at the aggregate level of the planning framework proposed here, as well as for more detailed studies of particular water systems. Models for water planning that have been used within the multiobjective framework, and that are of the same nature in many respects as the central models in the planning framework proposed here, are described in Major and Schaake (1972) and de Lucia and Rogers (1972). We recommend this approach for the planning process described below.

3.4 Proposed Water Resources Planning Process

3.4.1 Introduction: The purpose of this section is to provide donor nations and organizations and Sahel-Sudan nations and organizations with a process that will enable them to decide how to allocate their financial resources to detailed water resource studies concerned with long term development. We think it is important that these allocations for detailed studies be made only after either the entire region or an entire nation has been analyzed. The reason for this is that if it is not done, important investment opportunities might be missed. By examining large areas, it is also possible to determine the tradeoffs between investment in different parts of an area, which is valuable information to decision makers.

An interactive and iterative planning process should be used. An interactive process is needed so that planners can perceive the goals of the decision makers and decision makers can respond to planning results. Iteration is needed because planners cannot immediately perceive the goals of the decision makers and, more importantly, decision makers cannot initially state their goals into easily translated planning objectives. Decision makers need to see the consequences of various forms of actions before they can really be sure of their goals and choose the appropriate form of action or decision. Therefore, planners and decision makers should meet many times during a planning process. At each meeting the planners should present and explain alternatives to the decision makers for the decision makers' response. The alternatives presented should be based upon the results of previous meetings. During the initial meetings, planners should present broad alternatives representing a wide range of objectives. The later alternatives presented should perhaps be defined more narrowly.

The planning process described in the next two sections should be seen as a normative planning process. It would be impractical to expect that in any real planning situation all of the elements of the process suggested here would be reproduced exactly; yet, the process provides an important normative framework toward which the organization of planning institutions and methods can be directed.

3.4.2 <u>Planning Process - Donor Group - Recipient Group:</u> Ideally, the demand and supply models would be used in a planning process in which a group of international donor organizations is working with a group of recipient organizations. In this case, the planners would probably be a large consulting firm, and the clients the donor and recipient organizations.

1. The process would start by the donors meeting to learn how much long term aid each is willing to provide, and any prior commitments that have been made. They would also determine how

much money was available each year. The recipient group would also meet and decide upon the levels of human, industrial, agricultural, and livestock demands in each subarea in each country. This could be done in conjunction with the consultants who have previously divided the region into subareas and prepared the basic hydrologic data. To translate production targets and anticipated population growth into water needs, the demand model could be used. The net benefit formulation of the supply model could also be used to decide upon demand levels. The recipients would also determine how much money they propose to spend and how much is available each year. By being encouraged to specify target demand quantities instead of project lists, the recipients are forced to be explicit about production and human service target levels and, hence, national goals.

The donors and recipients would also discuss the allocation of the operation, maintenance, and replacement (OMR) and distribution system costs among the donors and recipients, and repayment procedures for any loans.

The consultants would also have to learn from the recipients the social and political constraints pertaining to the development of any source so that the appropriate constraints could be added to the supply model.

2. The consultants run the supply model given these demands and determine the cost and configuration as well as the cost breakdown by subarea. Runs minimizing the total cost as well as OMR costs would be made. If the demand set is infeasible because of hydrologic constraints (for example, groundwater recharge), the hydrologic condition (or conditions) would be relaxed until feasibility occurred. Using the original hydrologic conditions, the demand pattern could also be altered to achieve feasibility. Several variations of relaxed hydrologic constraints and decreased demands would be run. The consultants would

perform sensitivity analysis on the solutions to determine which development plans are sensitive to questionable data and the trade-offs between designing for several design years. They would also make an effort to determine the reliability of different parts of the total supply system based upon subarea recurrence intervals (or regional recurrence intervals if strong cross-correlations exist). Finally, the consultants would determine the approximate dollar benefits necessary for these demands to correspond to positive net benefits. This would be done using the procedure described in Section 2.5. The consultants would also determine the total distribution costs and add them to the supply source costs to determine the approximate total construction cost and total OMR cost.

3. A meeting would be held between donors and recipients to discuss the modelling results and implications. If the costs of developing the sources and distribution systems are less than or approximately equal to the donors' total estimated contributions and recipients shares less some percentage for additional studies and data gathering; if the actual benefits for each water user in each subarea seem plausible, and if the plans are agreeable to donors and recipients, work can proceed on the scheduling of further studies, data gathering, and project construction. However, if not all of these conditions are met and immediate compromise is not possible, an iterative procedure is necessary until they are all met. For example, if the demands exceeded the financial resources of the donors and/or the recipients, an effort must be made by the consultants to help the donors and recipients decide which demands to cut back. If some of the implied benefits from use are extremely low or questionable, the corresponding demands may be removed. The recipients could also be asked to redefine their development priorities in terms of the total budget available. One way to help them do this would be to run the supply model in a budget constrained,

net benefit maximization form. Either previously determined or new benefit values could be used depending upon the realism of the previously determined benefits. The consultants could also carefully examine the demand figures with the recipients to determine if they are reasonable, particularly the growth rates and target quantities. Consultants could meet with the donors to determine if they can increase their funding commitments. The donors could be presented with information on the extra funding required to meet the original demands or a reduced set that is still above their original commitment level. The donors would make their decisions based upon the benefits lost as a result of unsatisfied demands. If a large increase in benefits is possible for a reasonable increase in funding, the donors may be convinced to increase their financial commitments.

It is anticipated that by using such procedures as these, all the conditions can be met and an agreed upon set of preliminary supply sources and demands determined. Also agreed upon would be areas where more data is needed before final decisions can be made.

4. The consultants would make a first cut effort at scheduling the development of possible supplies and the further data studies. This scheduling could either be done heuristically or else by using a mathematical model. The model would be a sequencing model such as developed by Cohon et al. (1973) for Argentina. This model uses data on the availability of funds and manpower over time and the time variation of benefits to determine when each project and use should be scheduled.

The scheduling of data studies would be indirectly determined by the sequencing model, since these studies should precede the times when the supplies that are sensitive to the questionable data are scheduled to be on line. Obviously, before this time, the data uncertainties must be satisfactorily resolved.

5. A donors-recipients meeting would be held to discuss the consultants' schedule of the development of supplies and data studies. Adjustments would be made as necessary.

6. The last procedure is a continuous one. Donors-recipients meetings would be held regularly (perhaps semi-annually or annually) with the consultants to update the work. Any of the following may result in a change in the present project and data study list and schedule:

1. Results of data gathering activities
2. Changes in the funding situation of donors or recipients
3. Changes in the world market - particularly prices and supplies
4. Changes in national priorities.

All of these changes would be incorporated into the demand, supply and sequencing models and the results fully discussed by the donors and recipients. Changes may then be made in the future development of sources and uses, data gathering, and the schedule of these activities.

3.4.3 <u>Planning process: one donor and collection of recipients:</u> In this process, a donor is working with a group of recipients. However, each member of the group works separately with the donor. Such a situation may arise when the level of international cooperation described in the previous section is not possible. In this process, it is assumed that the donor would have control over the mathematical models through a consultant. The supply model would help the donor work with each recipient separately while still being aware of its regional responsibilities so that for example, in the extreme, all its aid is not distributed to only one or two recipients.

The process described below is less detailed than that in Section 3.4.2 because some of the details are the same.

1. The donor would decide how much money it has available on an annual basis.

2. The donor would learn separately from each recipient its projected demands. This could be done either through the use of consultants as in Section 3.4.2 or by just accepting the recipients' estimates. Social and political development constraints will also have to be determined.

3. The projected demands would be run on the supply model by the consultant, as done in Step 2 in the previous section.

4. The donor should determine in a preliminary manner what supply source development and data gathering activities it wishes to fund. This would be done by determining the benefits of each demand and running the supply model in the net benefit maximization format subject to budget constraints or simply by examining the implied benefits of the sources. There may also be model constraints on how much money is to be spent for each recipient. These could also be included in the model. The donor should remain in contact with other donors during this step, so that duplication of aid is avoided. A sequencing procedure is also used at this step to determine how the donor activities will be allocated over time once its activities have been selected.

5. A meeting is held between the donor and each of the recipients. Discussed at that time are the activities the donor wishes to undertake, and why. The donor's activities will be adjusted, based upon the recipients' views. This may require using the supply and sequencing models again. The donor may also discuss with the recipient its evaluation of the recipients' demands and their implied benefits.

6. The donor starts its supply source development studies and data gathering activities, maintaining contact with other donors.

7. A continuous planning procedure is set up between the donor and each recipient to insure the donor's funding activities can be adjusted to changes.

This process might well be less efficient than the international process described in Section 3.4.2. The major possible sources of inefficiencies are the donor nation trying to coordinate its activities with other donors and other recipients. Such inefficiencies may lead to duplication of aid, the non-support of favorable activities because each donor thinks the other is supporting them, and not taking full advantage of regional opportunities. Some of these inefficiencies could be lessened if the recipients interacted strongly among themselves and other donors.

3.4.4 Summary: Two possible iterative, interactive planning processes for long term water resource development in the Sahel-Sudan region have been described. The processes rely upon planners, donors, and recipients working together to determine suitable water uses and supply sources that reflect budget conditions. Periodically, the selected water uses and supply sources are reviewed to incorporate new data, funding changes, world market change, and changing national priorities. The process relies heavily upon a mathematical supply model to help select projects subject to social and political constraints. The use of a demand forecasting and scheduling model is also recommended. Obviously, these processes described are not inflexible, and variations are possible. The arrangements between donors and recipients are also not limited to the two described.

APPENDIX A

APPENDIX A

Users manual for the computer program projecting water needs: (followed by program listing)

1. Definition of Parameters and Their Dimensions

QTY (4, 30, 10):	Quantity level of economic or demographic sector for 4 time periods [1970, 80, 90, 2000].
WCOEFF (4, 30, 10, 10):	Water use coefficient for 4 time periods; 30 sectors; 10 countries; and 10 regions.
R ALL (4, 30, 10, 10):	Regional allocator
REGDF (4, 30, 10, 10):	Regional distribution factor
R QTY (4, 30, 10, 10):	Regional quantity
RWRQ (4, 30, 10, 10):	Regional water requirement.

N1 = Number of Countries

N2 = Maximum Number of Regions in One Country

N3 = Number of Sectors

N4 = Number of Time Periods

2. Input of variables: N_1, N_2, N_3, N_4

Input of parameter QTY for each country (1) by time period (L) and by sector (K).

3. DO I = 1, N_1 provides a printout for quantities within a format where the arrangement of QTY values will be the following:

The innermost loop arranges data by time for time periods 1-4, Loop K = 1, N_3 arranges data by sector:

Sector	1970	1980	1990	2000
1				

Sector (cont.)	1970	1980	1990	2000
2				
.				
.				
.				
N_3				

I = 1, N_1 loop assures the periodicity by country so the above table will be printed for each country.

4. Input of the other parameters within 3 nested loops:

a. Regional allocator (R ALL [L, K, I, J]) where the number of allocators is defined by the number of time periods (L), the number of sectors (K) and the number of regions (J) within each country (I). Number of allocators = L x K x J x I

b. Water use coefficient (WCOEFF[L, K, I, J])

c. Regional distributor (REGDF. [L, K, I, J])

The number of the parameters (WCOEFF and Regional distributor factor) is defined the same way as it is described in paragraph 4. a.

5. DO I, K, L, J arranges the printout of the three above mentioned variables. The innermost loop arranges the three variables by region J = 1, N_2; the L = 1, N_4 loop by time, the K = 1, N_3 loop by sector, and the I = 1, N_1 loop by country. This arrangement results in the following table:

			REGION				
Country	Sector	Time	1	2	3	4	5 - - - - - -
		1					
		2					

(cont.)

Country	Sector	Time	REGION 1	2	3	4	5 - - - -
	1	3					
		4					
	2	1					
		2					
		3					
		4					

6. The main program calls subroutine PROJECTION in order to project "quantities" for future decades. The subroutine is defined by the same parameters and the same variables as the main program itself. (QTY, WCOEFF, R ALL, . . . etc.) The variables have the same dimensions as they had in the main program.

The three "do" loops 5, 6, 7 provide the frame for the following calculations:

1. ABSVAL = ABS (QTY [L, K, I]) taking the absolute value of quantities (or growth rates).
2. If ABS VAL GT 10.0 GO TO 7

   This step checks the quantity data to determine whether or not they are absolute numbers or growth rates. If the number is greater than 10 (it is not a growth rate) the program skips the rest of the steps within loop 7 (time loop) and returns to check the next value that is, to the value of time period 3.

If the number is a growth rate (less than 10) the subroutine will project a value by substituting the quantity of the previous time period (QTY, [LL, K, L]) where LL = L-1, and L is the number of time periods into the following exponential equation:

QTY (L, K, I) = QTY (LL, K, I) x (EXP (RATE)) which is analogous to the $X_n = X_{n-1}$ actual increment (or growth rate of present time period) equation.

The next "write" statement prints the projected quantities by time period (2, 3, 4), Sector and Country, and this results in the following format:

Country	Sector	Projected Quantity	Time Period
	1		1980
			1990
			2000
	2		1980
			1990
1			2000
	3		

After the above listed steps are executed for country 1 (loop [5])
for sector 1 (loop [6])
and for time period 2 (innermost loop [7]),

the routine returns to the next value of the innermost loop (time 3) and projects quantity in the same way as earlier for the same country (country 1) and the same sector (sector 1), but for the next time period (time 3). When all the three projections are done for the 1st sector of the first country, the routine returns to Loop 6, increases the sector parameter by 1, and follows through the above calculations for Sector 2 in country 1. After projections are made for all time periods and all sectors within the 1st country, the routine returns to the outermost loop (5), increases the country parameter by 1 and starts the process again from the very beginning for country 2. This cyclical calculation will continue until all projections are made for each country. Then the program returns to the next step of the main program.

7. The following statement of the main program calls subroutine MULTIPLIER. This subroutine is defined by the same parameters and the same variables (common statement) as the main program and as subroutine PROJECTION.

There is a four-looped do statement performing three simple computations:

For each time period	(DO L = 1, N_4)
For each region	(DO J = 1, N_2)
For each sector	(DO K = 1, N_3)
For each country	(DO I = 1, N_1)

These calculations are the following:

Regional "quantity" = Total "quantity" of the country times Regional Allocator

Regional water requirement = Regional "quantity" times Water Use Coefficient

Re-distributed Regional water requirement = Regional Water Requirement times Regional Distribution Factor

The subroutine executes these computations for country 1, sector 1, region 1, and time period 1, then returns to the beginning of the innermost loop and performs the calculations for time periods 2, 3, 4. After completing the time loop, the regions loop is next. The computation will be done for country 1, sector 1, region 2 for all the four time periods. When the routine arrives at the next loop, it increases the value of sector-parameters (K) by 1 and performs the calculations for country 1, sector 2, regions 1 to 10 and time sequences 1 to 4 for each region. The last loop is the outermost one, therefore every time the value of country-parameter (I) is increased by 1, all the inside-loops will be operating in the same sequence as described above, performing the required computation.

8. Returning to the main program, a write statement orders the print-out of the results calculated by subroutine MULTIPLIER in the following format:

Country	Region	Sector	Time	Regional Water Requirement	Redistributed Regional Water Requirement
1	1	1	1		
1	1	1	2		
1	1	1	3		
1	1	1	4		
1	1	2	1		

```
               SUBROUTINE PROJE (N1,N2,N3,N4,N5)
               COMMON QTY(4,30,10),WCOEF(4,30,10,10),RALL(4,30,10,10),
              X REGDF(4,30,10,10),RQTY(4,30,10,10),RWRQ(4,30,10,10),
              X DRWPO(4,30,10,10)
               JQ=0
               WRITE(6,2)
               DO 5 I=1,N1
                  DO 6 K=1,N3
               WRITE(6,1)
                     DO 7 L=2,N4
                     ABSVAL=ABS(QTY(L,K,I))
         C    CHECK TO SEE IF CPOP IS RATE OR NUMBER
                     IF(ABSVAL.GT.10.0)  GO TO 7
                     LL=L-1
                     RATE=QTY(L,K,I)
                     QTY(L,K,I)=QTY(LL,K,I)*(EXP(RATE))
               IF (L-3)10,20,30
         10    WRITE(6,11) I,K,QTY(L,K,I)
               GO TO 8
         20    WRITE(6,21) I,K,QTY(L,K,I)
               GO TO 8
         30    WRITE(6,31) I,K,QTY(L,K,I)
         8     JQ=JQ+1
               IF(JQ.LT.39)GO TO 7
               JQ=0
               WRITE(6,2)
         7           CONTINUE
         6        CONTINUE
         5     CONTINUE
         1     FORMAT(9X,'---------------------')
         2     FORMAT('1',1X,'ESTIMATED QUANTITIES FOR VARIOUS TIME SEGMENTS'/
              X1X,'COUNTRY',2X,'SECTOR',20X,'VALUE',2X,'TIME'/)
         11    FORMAT(5X,I2,6X,I2,F26.3,2X,'1980')
         21    FORMAT(5X,I2,6X,I2,F26.3,2X,'1990')
         31    FORMAT(5X,I2,6X,I2,F26.3,2X,'2000')
               RETURN
               END
```

```
0040              DO 400 J=1,N2
0041              DO 400 K=1,N3
0042              IF(JJ.LT.50)GO TO 405
0043              WRITE(6,500)
0044              JJ=0
0045        405     IF(K.GE.19) GO TO 430
0046                IF(K.EQ.18) GO TO 440
0047                 IF(K.LE.2) GO TO 450
0048               SUM2=SUM2+RWRQ(L,K,I,J)
0049              WRITE(6,100) I,J,K,L,RWRQ(L,K,I,J),DRWRQ(L,K,I,J)
0050              JJ=JJ+1
0051              GO TO 400
0052        430   SUM4=SUM4+RWRQ(L,K,I,J)
0053               GO TO 490
0054        440   SUM3=SUM3+RWRQ(L,K,I,J)
0055               GO TO 490
0056        450   SUM1=SUM1+RWRQ(L,K,I,J)
0057        490   CONTINUE
0058        600   WRITE(6,101) I,J,K,L,RWRQ(L,K,I,J),DRWRQ(L,K,I,J)
0059              JJ=JJ+1
0060        400   CONTINUE
0061               WRITE(6,105)I,L,SUM1,SUM2,SUM3,SUM4
0062        105   FORMAT(//1X,'TOTAL FOR COUNTRY  TIME  SECTORS',
                 X/1X,I14,I8,3X,'1TO 2  ',F28.2, 4X,'LITER/DAY',
                 X/26X,'3TO17  ',F28.2, 4X,'LITER/YEAR',
                 X/27X,'  18  ',F28.2,4X,'LITER/DAY',
                 X/27X,'19TO24',F28.2,4X,'LITER/DAY',//)
0063        420   CONTINUE
0064        410   CONTINUE
0065        10    FORMAT(5I2)
0066        11    FORMAT(4G20.8)
0067        12    FORMAT(10F8.3)
0068        100   FORMAT(3(6X,I2),4X,I2,10X,F20.2,5X,'LITER/YEAR',5X,F20.2)
0069        101   FORMAT(3(6X,I2),4X,I2,10X,F20.2,5X,'LITER/DAY ',5X,F20.2)
0070        201   FORMAT(2(5X,I5),5(5X,E10.4))
0071        500   FORMAT('1',50X,'WATER NEED',20X,'CORRECTED WATER NEED'/
                 X1X,'COUNTRY',2X,'REGION',2X,'SECTOR',2X,'TIME',23X,'(LITER)',
                 X33X,'(LITER)'/)
0072        505   FORMAT('1',20X,'TIME',102X,'DATA'/1X,'COUNTRY',2X,'SECTOR',2X,'SEG
                 XMENT',1X,'REGION 1',2X,'REGION 2',2X,'REGION 3',2X,'REGION 4',2X,
                 X'REGION 5',2X,'REGION 6',2X,'REGION 7',2X,'REGION 8',2X,'REGION 9'
                 X,2X,'REGION 10',2X,'TYPE'/)
0073        510   FORMAT('1',3X,'COUNTRY',4X,'SECTOR',10X,'GROWTH RATE OR ESTIMATED
                 XQUANTITIES'/36X,'1970',10X,'1980',10X,'1990',10X,'2000'/)
0074        801   FORMAT(2(3X,I5),4X,I5,1X,10F10.3,1X,'RALL')
0075        802   FORMAT(2(3X,I5),4X,I5,1X,10F10.3,1X,'WCOEF')
0076        803   FORMAT(2(3X,I5),4X,I5,1X,10F10.3,1X,'REGDF')
0077              END
```

```
      C        DEFINITIONS OF VARIABLES
      C
      C     N1= NUMBER OF COUNTRIES          I IN THE GENERAL USE
      C     N2= NUMBER OF REGIONS             J IN THE GENERAL CASE
      C     N3= NUMBER OF SECTORS            K IN THE GENERAL CASE
      C     N4= NUMBER OF TIME SEGMENTS       L IN THE GENERAL CASE
      C     N5= LENGTH OF TIME PERIOD
      C
      C        IMPORTANT NOTE
      C         SEQUENCING MODEL MAY BE ENTERED AS A CURRENT FIGURE AND GROWTH RATES
      C
            COMMON CTY(4,30,10),WCOEF(4,30,10,10),RALL(4,30,10,10),
           X REGDF(4,30,10,10),RQTY(4,30,10,10),RWRQ(4,30,10,10),
           X DRWRQ(4,30,10,10)
            JJ=0
            JC=0
            JI=0
            READ(5,10)N1,N2,N3,N4,N5
            READ(5,11) (((QTY(L,K,I),L=1,N4),K=1,N3),I=1,N1)
      C
            WRITE(6,510)
            DO 200 I=1,N1
            DO 200 K=1,N3
            WRITE(6,201) I,K,(QTY(L,K,I),L=1,N4)
            JI=JI+1
            IF(JI.LT.50)GO TO 200
            WRITE(6,510)
            JI=0
        200 CONTINUE
            READ(5,12)  (((( RALL(L,K,I,J),J=1,N2),L=1,N4),K=1,N3),I=1,N1)
            READ(5,12)  ((((WCOEF(L,K,I,J),J=1,N2),L=1,N4),K=1,N3),I=1,N1)
            READ(5,12)  ((((REGDF(L,K,I,J),J=1,N2),L=1,N4),K=1,N3),I=1,N1)
            WRITE(6,505)
            DO 300 I=1,N1
            DO 300 K=1,N3
            DO 300 L=1,N4
            WRITE(6,801) I,K,L,( RALL(L,K,I,J),J=1,N2)
            WRITE(6,802) I,K,L,(WCOEF(L,K,I,J),J=1,N2)
            WRITE(6,803) I,K,L,(REGDF(L,K,I,J),J=1,N2)
            JC=JC+3
            IF(JC.LT.50)GO TO 300
            WRITE(6,505)
            JC=0
        300 CONTINUE
            CALL PROJE (N1,N2,N3,N4,N5)
            CALL MULTI (N1,N2,N3,N4,N5)
            WRITE(6,500)
            DO 410 L=1,N4
            DO 420 I=1,N1
            SUM1=0
            SUM2=0
            SUM3=0
            SUM4=0
```

```
0001                 SUBRCUTINF MULTI (N1,N2,N3,N4,N5)
0002                 COMMON QTY(4,30,10),WCOEF(4,30,10,10),RALL(4,30,10,10),
                    X REGDF(4,30,10,10),RQTY(4,30,10,10),RWRQ(4,30,10,10),
                    X DRWRQ(4,30,10,10)
0003                 DO 30 I=1,N1
0004                    DO 31 K=1,N3
0005                       CO 32 J=1,N2
0006                          DC 33 L=1,N4
0007                          RQTY(L,K,I,J)=QTY(L,K,I)*RALL(L,K,I,J)
0008                          RWRC(L,K,I,J)= RQTY(L,K,I,J)*WCOEF(L,K,I,J)
0009                          DRWRQ(L,K,I,J)=RWRQ(L,K,I,J)*REGDF(L,K,I,J)
0010           33             CCNTINUE
0011           32          CCNTINUE
0012           31       CONTINUE
0013           30    CONTINUF
0014                 RETURN
0015                 END
```

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