UNITED NATIONS DEVELOPMENT PROGRAMME
WORLD HEALTH ORGANIZATION

GOVERNMENT OF SURINAM

PUBLIC WATER SUPPLIES AND SEWERAGE PROJECT

VOLUME IV
WATER SUPPLIES AND SEWERAGE

REPORT PREPARED BY THE
WORLD HEALTH ORGANIZATION
ACTING AS EXECUTING AGENCY
AND THE UNITED NATIONS ACTING
AS PARTICIPATING AGENCY FOR
UNITED NATIONS DEVELOPMENT PROGRAMME

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Report prepared by the
World Health Organization
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PREFACE

Operations started on the project "Public Water Supplies and Sewerage (NET-4)" with the arrival of the Project Manager in September 1969. The Plan of Operations was signed by the Government of Surinam, the World Health Organization, and the United Nations Development Programme on 30 October 1970. The cooperating government agency was the Ministry of Rural Government and Decentralization. The World Health Organization was named Executing Agency and was assisted by the United Nations in the capacity of Participating Agency, by virtue of a Standard Letter of Agreement which was signed by the agencies in July 1970.

It is noted that no provision was made to contract for the consulting engineering services in the preparation of the engineering studies, reports, and designs. In this departure from the usual World Health Organization practice in UNDP(SF) preinvestment projects, the responsibility for the above tasks rested with the international professional staff assigned to the field activity of the project.

This final report represents a statement which descriptively, qualitatively, and comprehensively summarizes the findings and recommendations of the project team, incorporating the public water supplies, sewerage, and water resources investigation aspects of the project. It comprises the following four volumes:

Volume I - Summary Report
Volume II - Basic Data and Special Studies
Volume III - Water Resources (Hydrogeological and Hydrological Studies)
Volume IV - Water Supplies and Sewerage

The subject volume, Volume IV, covers water supplies and sewerage and includes general considerations, design bases for water supplies, water quality requirements, sources of supply, water treatment, interim water supplies, and the recommended water supply program.

Conferences and discussions were held with and direct assistance obtained from:

Ministry of General Affairs
Ministry of Health
Ministry of Public Works
Ministry of Agriculture
Ministry of Finance
Ministry of Development
Ministry of Rural Government and Decentralization

In March 1970, a mid-project review mission visited the project. The mission included the following:

Ross Milley	Assistant Regional Representative, UNDP, Trinidad and Tobago
G. D. Soerdjoesingh	Project Coordinator, Ministry of Rural Government and Decentralization
A. Jap Tjoen San	Surinam UNTAB Liaison Officer
Paul Bierstein	Chief, Preinvestment Planning, WHO, Geneva
Harry G. Hanson	Regional Advisor, Engineering and Environmental Sciences, PAHO/WHO, Washington
Whitman C. Dimock	Technical Advisor, Resources and Transport, Division of Economic and Social Affairs, United Nations, New York
John T. Robinson, Chairman	Zone Engineer, PAHO/WHO, Zone I Office, Caracas

The field operations were conducted by a team of United Nations specialized agency staff and Surinam Government staff. The team was structured as follows:

Permanent Field Staff

International Staff

World Health Organization:

S. G. Serdahely, Project Manager
J. G. Copley, Project Manager
J. L. Vincenz, Project Manager
C. L. Philipovsky, Sanitary Engineer (Waste Water)
R. J. Pitters, Sanitary Engineer (Water Supply)
Mrs. Christina Kambel, Secretary

United Nations:

V. R. Dixon, Hydrogeologist
C. K. Stapleton, Drilling Superintendent

Surinam Government Counterpart Staff

G. D. Soerdjoesingh, Project Coordinator

Engineers

R. Dihal
R. Randjietsingh
E. T. Tsai Meu Chong
R. Nanden
S. Autar

Surveyor

R. Biharie

Short-term Consultants

C. Clinton Davis, Management
D. Duba, Hydrology
C. N. Stutz, Industrial Waste
L. Huisman, Biological Filtration
Jean L. Vincenz, Management
Donald E. Crum, Water Waste

Visiting Specialists

M. Suleiman, PIP, WHO, Geneva
W. C. Dimock, UN, OTC
E. Elmore, ES, PAHO, Washington
O. Cordero, Fluoridation, PAHO, Washington
D. J. Williams, Fluoridation, Canada

Non-professionals

A. Ghafoerkhan
J. Orie
J. Ragoebar
H. C. Faerber
H. Rambalie
S. Ramdat
H. A. Khodabaks

Drilling Supervisors

A. Staphorst
P. Viereck
W. Bouman
S. Ramdas
M. Petricie
J. Doornkamp
E. Wilson

All of the agencies listed below, their representatives, and individuals readily contributed to the implementation of this project in an atmosphere of mutual assistance for the benefit of Surinam:

Army of the Kingdom of the Netherlands
Center of Agricultural Research in Surinam
Surinam Water Company
Surinam Aluminum Company
Billiton Company of Surinam
Mariënburg Sugar Enterprise
Bruynzeel Surinam Wood Company
Representatives of the Dutch Five-Year Plan
Representatives of several United Nations specialized agencies

It is with sincere appreciation that their efforts, as well as those of many others who also brought their ideas, comments, experience, and wisdom in generous and unsparing measure, are gratefully acknowledged.

During the long dry season of 1971, conditions developed in which the water supply system of the city of Paramaribo was unable to meet the demands, and this resulted in loss of production as well as the retrogression of service from continuous to intermittent. Aware of the possibly serious consequences of these developments, the Government requested the assistance of the incumbent staff of the UNDP(SF) project in effecting the necessary improvements to the water supply as well as to provide the investigation and planning required to meet present and future needs for the city system. As a result, the UNDP(SF) project was extended from the original three-year span to cover a 42-month period. The report incorporating the Paramaribo study is a separate and supplementary volume. A second six-month extension was requested by the Government for surface drainage studies. The report for these studies is also a separate and supplementary volume.

The metric system of weights and measures is used in Surinam for engineering purposes generally. However, standard practice in the design of water supply systems in the country has been to express pipe sizes in inches and to express pump capacities in U.S. gallons per minute. Accordingly, these units have been used in designs.

In estimating costs the currency used is the Surinam guilder (Sf.). The rate of exchange current when estimates were prepared during project operations was Sf.1.87 to US$1.00.

All elevations indicated in this report are referenced to the official NSP (Normal Surinam Level) datum. Established in 1958, NSP is based on the average sea level, measured at an automatic sight gage structure on the Suriname River at Purmerend Plantation, located near Paramaribo at Leonsberg. Elevations referred to an earlier datum, SP, are converted by Surinam authorities through the application of a negative 8.00 meter factor.

Acronyms used in this report are included in the glossary at the end of Volume I.

CHAPTER 1

WATER SUPPLY - GENERAL CONSIDERATIONS

INTRODUCTION AND BACKGROUND

For several years during the 1950's, the U.S. Agency for International Development through the Surinam-American Technical Cooperation Service (SABTS) conducted a demonstration project for developing rural water supplies through the installation of low-yield, sanitary wells, equipped with hand-operated pumps. This project created considerable interest in water supply development and led to the SABTS program of supplying technical assistance, consultation services, and training measures. The general objective was to encourage the Government of Surinam in assuming the responsibility for water supply development and administration, with technical assistance being provided by USAID and SABTS.

In 1962 the Director of Public Works indicated his interest in establishing a water supply section within his Ministry which would be responsible for promoting or developing water supply improvements throughout the country and requested SABTS assistance in drafting a plan which could be submitted by the Minister of Public Works to the Cabinet of Ministers for consideration and approval. Subsequently, a plan was submitted to the Director of Public Works that provided for establishing within the Ministry a water supply section which eventually might be converted into an autonomous water supply authority. The plan called for the Water Supply Section to receive the benefits of 1) a high-level advisory committee, 2) a health educator whose services would be made available by the Director General of Public Health, 3) experienced personnel and water supply equipment from SABTS, and 4) professional advisory services and assistance in personnel training from USAID.

However, the SABTS assistance as well as that of USAID was phased out during the mid-1960's, and the Government elected to pursue the UNDP(SF) project route for technical assistance in meeting the problems of water supplies as well as sewerage systems in Surinam. Initial efforts to obtain this assistance were commenced in 1964, with the cooperation of the PAHO/WHO Zone Office. These efforts eventually led to the fielding of the Preparatory Assistance Mission Team, as previously discussed in the "History of the Project."

The main findings and recommendations of the Preparatory Assistance Mission are quoted as follows:

"Currently, the most economical source of potable water supply is ground water. Present evidence indicates that such water of acceptable quality should be available in sufficient quantity to meet the future needs. Some problems of salt water intrusion are, however, already being experienced in certain wells (in the Zorg en Hoop well field in Paramaribo), and other wells may turn saline as the rate of extraction is increased to meet the rapidly growing demands for water. The rates of replenishment of the aquifers have yet to be investigated in detail.

"The withdrawal of additional potable ground water should not adversely affect irrigation, hydropower and navigation development based on surface water, but future impounding of surface water in the basin for hydropower or irrigation storage might, if not coordinated, drastically affect the rate and salinity of recharge to ground water storage. Unified water management is therefore required.

"If, eventually, the ground water supply becomes inadequate, it can be supplemented by surface supplies; but the costs of treatment and transmission of surface supplies would be much higher than those now being incurred for ground water supplies.

"It is inadvisable to commit further investment to ground water withdrawal until the characteristics of the aquifers in the lower Suriname River basin have been investigated. This investigation is a prerequisite to the planning of new water supply installations in Group I area. The findings should be helpful in defining the extent of the subsequent investigations required for the provincial coastal areas.

"The progress on the Government's water supply and sewerage development programme is dependent on the progress of these hydrogeological investigations, and only the minimum of specialist expertise is needed to enable investigations to start. Therefore, it is recommended that the preparatory work should start not later than 1 July 1969. Additionally, it is hoped that the outcome of the IACB meeting in October 1969 will result in immediate authorization to incur expenditure of Special Fund monies for the remaining equipment required by the Project, so that the Project can be accelerated to a fully operational basis as soon as possible. The Government of Surinam has allocated US$92,000 from the 1969 budget and has also made all arrangements for the use of equipment and office space."

These findings were endorsed in general principle with minor exceptions by the UNDP(SF) project team, with regard to the scope of activities as well as the geographical areas included. As is the case in studies of this nature, additional investigations became necessary, and these were implemented in addition to the original tasks in order to provide as comprehensive a picture as possible.

OBJECTIVES

The objectives, therefore, of the immediately following sections of this report are to develop engineering proposals leading to the eventual provision of consistently available quantities of potable water to meet all reasonable domestic requirements as well as the concurrent commercial and industrial needs, while providing a degree of fire protection. Volume III - Water Resources, delineates the hydrogeological investigation program as well as the hydrological studies and findings.

In order to develop the above proposals it was necessary to determine suitable design allowances (on a per capita or area basis) to meet the present and future needs of the Supply Group areas. Following the establishment of design criteria, total water quantities and quality requirements were determined to the degree possible and, whenever possible, definite suitable services selected.

Existing water supply and distribution facilities were studied in depth with a view to determining the capabilities for meeting design criteria and also with respect to eventual standardization of methods.

Finally, the engineering proposals were developed and are presented according to a schedule of priorities which is designed primarily to provide relief in areas of most immediate need, and also to permit prudent phasing of construction.

PRESENT CONDITIONS AND EXISTING WATER SUPPLY FACILITIES

Within the organizational framework of Surinam's central government, the Ministry of Public Works originally had the responsibility for developing sources of potable water and for distributing it for use by the people of Surinam. To accomplish this task, the Ministry depended on appropriations of funds from the national budget, except for those established systems which were operated under contract wherein revenue realized from the sale of water was sufficient to offset the cost of making piped water service available to the consumers.

In spite of the magnitude and importance of its responsibilities in this field, the Ministry did not thoroughly develop within its own organization the technical facilities and trained personnel required to plan, install, operate, and manage water supply systems. In early years projects undertaken directly by Public Works personnel with respect to water supply and distribution were essentially limited to:

1. Constructing rain water catchments and collection reservoirs.

2. Installation in rural areas of hand-dug wells of the type developed by SABTS.

3. Transporting water from available sources to people living in areas where stored quantities of fresh (rain) water were not sufficient to supply the demands of consumers during extended periods of dry weather. This transporting of water during dry seasons, by truck,

barge, or other available means, became a routine function of government and is still provided for in the budget of the Ministry of Rural Government and Decentralization.

It is acknowledged that this latter method of water distribution is costly; and the methods of handling expose the water to contamination, and thus to the likelihood of disease transmission. At best, it constitutes an emergency measure undertaken for the express purpose of alleviating chronic problems.

Traditionally the Ministry of Public Works is the governmental department for designing and building a wide range of public facilities such as buildings, roads, canals, sluice gates and lock installations, airports, and hydroelectric plants, but water supply systems and water treatment facilities were not included routinely among the projects justifying direct attention, except those small rainwater systems which in most cases are integral parts of building projects where the roof surfaces are utilized as catchment areas.

As the population of Paramaribo increased through the years, the need for a suitable community water supply system became more urgent. Non-government interests responded to the growing water demands by creating a water company, which was legally incorporated under the laws of the country. Legislative action in 1929 placed the corporate holdings in the hands of several stockholders which included banks, a trading company, and the Government of the Netherlands. While records indicate that the enterprise was originally initiated many years prior to that date, it is generally recognized that the operations of the Water Company on a substantial scale got under way in 1930 on the basis of the 1929 legislation. In 1948, the Government acquired all shares of corporate stock and became sole owner of the Surinam Water Company.

Following its inclusion as a new Ministry in the Government, the Ministry of Rural Government and Decentralization, established in November 1969 to succeed the Ministry of Public Works in this responsibility, is now the government representative and hence the sole stockholder of the Surinam Water Company. This provides the Ministers with an implied "veto" authority over actions taken by the Board. However, there is no indication that such authority has ever been exercised. In reality, the relations between the Government, represented by the Minister, and business interests, represented by the members of the Board, are well balanced, and this permits the Company, although totally owned by the Government, to function strictly as an autonomous business enterprise.

The Company plans, designs, constructs, operates, and manages urban water supply facilities. Projects for which the Company has assumed responsibility have been limited to areas with concentrations of population which the Company considers as justifying the installation of community water supply systems, involving pressure systems, treatment works (if indicated), storage facilities, and a piped distribution network.

Since it came into existence, the Surinam Water Company has had total responsibility for all water supply operations in Paramaribo (supply, distribution, and management). It has also undertaken water supply projects in other centers pursuant to contracts with the Ministry of Public Works.

In 1962 the plan referred to earlier was proposed by the Ministry of Public Works, SABTS, and USAID in which district water supplies would receive greater attention.

The plan provided for the Ministry of Public Works, through a proposed Water Supply Section, to be responsible for:

1. Development, operation, and maintenance of water supplies in rural areas. This would involve continuation of the program of water exploration and sanitary well construction previously carried out by SABTS as a demonstration project.

2. Promotion of improvements in piped water systems and the development of other "urban" systems to serve centers of population in the districts, on the basis of contracts between the Ministry and the Surinam Water Company. Similar contracts would authorize the Water Company to operate and maintain such water systems on a soundly established financial structure whereby revenue for water sold would be used exclusively for operating and improving water works installations. The plan foresaw that the Ministry of Public Works would be required to arrange for grants of funds to be used in financing water supply installations made by the Water Company on the basis of contracts with the Ministry. These grants might be available from the "10-Year Plan," the regular government budget, or from other possible sources, including loans.

In other words, the plan proposed that the Water Supply Section would be the authority within the Government responsible for negotiating contracts with the Surinam Water Company for undertaking development, operation, and management of community (urban) piped water supply systems, and that the rural water development (sanitary well) program would be carried out under the direct administration of the Water Supply Section. In some cases, the Water Supply Section through the Surinam Water Company could arrange to supply water to consumers on the basis of special agreements with others. The two principal bauxite mining companies operate water supply systems which serve their communities at Moengo, Paranam, and Onverdacht.

Small water supply systems are also being operated by the Ministry of Health and the Ministry of Agriculture.

Following formation of the Water Supply Section (now a part of the Ministry of Rural Government and Decentralization), the Surinam Water

Company has progressively turned over the smaller systems to that Section for maintenance and operation. At present, the Water Company retains the Paramaribo, Nieuw Nickerie, and Albina urban systems.

NEED FOR IMPROVEMENTS

Table IV-1 lists the existing non-private water supply systems in Surinam, together with the population served by the supplies. Each system is either being expanded or is under consideration for augmentation.

In most cases, water is supplied on a continuous 24-hour per day basis, but in some of the systems intermittent supply is necessary. The ever-present danger of contamination inherent in intermittent supplies is well known, and the World Health Organization maintains as a basic precept in design the requirement for a continuous supply of potable water.

The present inadequacies in water supplies are considered an important factor in contributing to environmental health problems, particularly in those areas where there tends to be a prevalence of gastrointestinal disease. More than 60% of the population lack safe piped water supplies and have to rely on untreated water. The use of questionable surface sources of water which may be polluted with pathogenic organisms should be reduced through the provision of safe, adequate, and readily available community supplies.

It is difficult to obtain reliable health statistics, and those that can be obtained are not always current. Table IV-2 provides a resumé of general health statistics, which have been collected from several sources. Mortality statistics are shown in Table IV-3.

Economic benefits will also derive through increased productivity following the elevating of standards of health by means of improved water supplies, thereby reducing absenteeism in industries, agriculture, and schools.

TABLE IV-1

EXISTING PUBLIC WATER SUPPLY SYSTEMS IN SURINAM

Supply Population Served	Source	Treatment	Storage	Distribution
Paramaribo (SWC) 175,000	20 shallow wells and 10 deep wells at Republiek 17 deep wells at Zorg en Hoop 4 deep wells at Leysweg (3 operating, 1 standby)	A,R,S	4,400 m^3: Republiek 4,000 m^3: Zorg en Hoop 700 m^3: elevated 18,000 m^3: Blauwgrond and Leysweg 1,000 m^3: Lelydorp	Approx. 400 km, mostly asbestos cement, some steel, and 2- to 12-in. PVC. Also 40 km 14-in. transmission from Republiek
Koewarasan (D and D) 2,000	2 deep wells	A,R	60 m^3: ground level 10 m^3: elevated	Approx. 18 km 2-, 3-, and 4-in. PVC.
Meerzorg (D and D) 6,000	2 deep wells	A,R,M	37 m^3: ground level 9 secondary of 8 m^3 each with handpump	Approx. 11 km of 2-in. PVC.
Wonoredjo (D and D) 2,000	Bulk purchase from Suralco	Total surface water treatment	300 m^3: ground level 12 m^3: elevated	Approx. 14 km of 2-in. PVC.
Albina (SWC) 2,000	2 shallow wells	A,R	100 m^3: ground level 10 m^3: elevated	Approx. 4 km of 4-in. and 2-in PVC.
Nw. Nickerie (SWC) 10,000	2 deep wells	A,R,S	300 m^3: ground level 40 m^3: elevated	Approx. 16 km of 6-in. AC pipe and 2-in. PVC pipe
Coronie (D and D) 3,000	1 deep well	A,R	70 m^3: ground level 8 m^3: elevated	Approx. 20 km of 4-in. AC and PVC and 2-in. PVC.

KEY:

A = Aeration
R = Rapid sand filtration
S = Shell filtration (pH adjustment)
M = Manganese filtration
C = Chlorination

SWC = Surinam Water Company
D and D = Ministry of Rural Government and Decentralization
ACP = Asbestos-cement pipe
PVC = Polyvinylchloride pipe

TABLE IV-1 (cont.)

EXISTING PUBLIC WATER SUPPLY SYSTEMS IN SURINAM

Supply Population Served	Source	Treatment	Storage	Distribution
Groningen - Tambaredjo (D and D) 2,200	2 deep wells	A,R	100 m^3: ground level 10 m^3: elevated	Approx. 12 km of 4-in. and 2-in. PVC.
Brownsweg (D and D) 3,000	Surface (lake)	C	Four tanks of 180 m^3 combined capacity	5 km of 2-in. galvanized iron pipes
Klaaskreek (D and D) 1,500	1 deep well	C	Reservoir on a hill, capacity 30 m^3	2.5 km of 2-in. PVC.
Brokopondo (D and D) 1,000	1 deep well	A,R	Two 3 m^3 pressure tanks and clear well of 70 m^3 capacity	2 km of 2-in. and 4-in. PVC
Kampong Baroe (D and D) 2,000	1 deep well	A,R	20 m^3: ground level 10 m^3: elevated	Approx. 11 km of 4-in., 3-in., and 2-in. PVC.
Alliance (Ministry of Agriculture) 450	1 deep well	A,S,R	Two 1.5 m^3	1.6 km of 4-in., 2-in., and 1-in. ACP and PVC.
Groot Chatillon (Ministry of Health) 150	Surface (river)	R,C	36 m^3 (filled twice a day)	Approx. 2 km of 4-in. and 2-in. asbestos cement

TABLE IV-2

GENERAL HEALTH STATISTICS

Population (estimated mid-year 1971)	408,100
Birth rate per 1,000 (1968)	32.8
Death rate per 1,000 (1968)	6.8
Natural growth rate, per cent (1968)	2.6
Infant mortality rate, per 1,000 live births (1968)	39.6
Government expenditure per capita for health sector (1970) US$	22.55
Per cent of total budget for health (1971)	10.0

TABLE IV-3

MORTALITY STATISTICS
(1968)

Disease	Rate per 100,000 Deaths
Typhoid, paratyphoid, and other salmonella infections	353
Bacillary dysentery and amebiasis	471
Enteritis and other diarrheal diseases	294
Bilharziasis (1967)	1,266

CHAPTER 2

BASES FOR DESIGN OF WATER SUPPLIES

GENERAL

Population studies conducted in the project area emphasize the predominantly residential character of the population zones. Because of this, the domestic water demands served as the primary basis for estimating individual water supply requirements. Since there is no substantial requirement for government and commercial establishments and industries, the allowance for these is calculated as a percentage of domestic allowances.

DOMESTIC REQUIREMENTS

It is noted that the Paramaribo metropolitan water supply system continues to be operated and maintained by the Surinam Water Company and is regarded as a separate entity as this report is drafted. As such it is not included in this report, except to provide data which contributes to the whole picture of public water supply in Surinam. The report covering the Paramaribo study is a separate document.

Average per capita requirements of water for small urban or rural communities are usually derived from studies of records of existing supplies and also the population served. The increase in per capita consumption is then projected in order to determine future design allowances.

In Surinam the usage quantities vary widely for the small systems which have been established for several years. For example, the average per capita usage for the Klaaskreek system is 10 l/c/d, while the average peak for Nieuw Nickerie is 204 l/c/d. Conditions other than normal usage tend to cause these extreme values to develop.

In view of the above and also because water consumption data in Surinam tend to be limited in scope and availability, the average per capita usage was estimated on the basis of judgement, conditioning the judgement in accordance with experience gained in other developing countries.

Consumers were divided into three categories as follows:

Category I : Occupants of dwellings supplied with a single Fordilla valve, but without metered service. (These valves are self-closing and will deliver only a certain given quantity of water for each opening of the valve.)

Category II : Occupants of dwellings supplied by a single tap in the dwelling and with metered service.

Category III: Occupants of dwellings which contain a variety of plumbing fixtures and have metered service.

It is noted that no special consideration is given to public standpipe usage. While this type of service exists to a minor degree in the country, the Government is seeking to discontinue this approach, providing instead the Fordilla valve service as the minimum acceptable. The Executing Agency concurs in this procedure, deeming it desirable to provide individual service wherever possible.

Category I - Consumers

This group of consumers is common to all small urban and rural supply systems in Surinam. A study of the ratio of consumer types showed that approximately 35% of the house services fit in this category.

Estimates for domestic per capita usages for the several years covered by this project are as follows:

Commissioning year	- 1972	30 l/c/d
15-year design year	- 1987	40 l/c/d
Ultimate plan year	- 2000	50 l/c/d

Category II - Consumers

Similar to the Category I group, these consumers constitute approximately 35% of the total number of house services in the existing water supply systems.

Estimates for domestic per capita usage are given as follows:

Commissioning year	- 1972	50 l/c/d
15-year design year	- 1987	70 l/c/d
Ultimate plan year	- 2000	90 l/c/d

Category III - Consumers

This group of consumers occupy dwellings which constitute approximately 30% of the total number of house services in the water supply systems.

Estimates for domestic per capita consumption are as follows:

Commissioning year	- 1972	70 l/c/d
15-year design year	- 1987	90 l/c/d
Ultimate plan year	- 2000	110 l/c/d

CONSUMPTION ANALYSIS

Obviously, not all the systems to be built in Surinam will be commissioned during 1972. This datum was established for the initial feasibility studies and served as a standard upon which comparisons for requirements of future system designs could be considered.

In addition to the estimates of the per capita usages for the above planning year categories, further consideration was given with respect to the probable changes within the categories which would obtain subsequent to the experience gained by users. In other words, it was anticipated that the progression from Fordilla valve service to single-tap service to "plumbed-house" service would take place. It was also assumed that, in the average water supply system, the number of connections would increase, because this pattern is typical for piped water supplies. With regard to the types of service, it was assumed that during the first 15-year period the total of "plumbed" houses would increase from 35 to 40%—a trend which is similar to that in Paramaribo. Thus, the remaining 60% would be fairly evenly divided among the Category I and Category II types of single-tap service. This situation would then be assumed to hold relatively constant until the ultimate plan year.

Further, it was estimated that the percentage of dwellings connected during the first year of operation of the water supply systems would be approximately 25%. This percentage would then be assumed to increase over the first 15-year period until 90% of the dwellings would be connected. Because this represents a practical maximum level, the percentage of houses connected was assumed to remain at 90% throughout the balance of the master planning period.

Per capita consumption including all uses and losses projected through the design cycles are shown in Table IV-4.

As shown in Table IV-4, the values for industrial, commercial, and institutional uses were estimated on the basis of probable growth in these sectors rather than on planned growth. Since no official development plan has been accepted by the Government as yet, the "industrial estate" concept cannot be applied; nor is there strict enforcement of zoning ordinances. Thus, the above values represent estimates only.

Water which is produced and sent into the distribution system but has not been sold, measured, or otherwise tallied is defined as unaccounted-for water and in this study was estimated at 20% of production, based on experience gained in similar countries. This figure is corroborated by Babbitt, Doland, and Cleasby, "Water Supply Engineering," McGraw-Hill Book Co., 1967. In the Paramaribo water supply system the amount of unaccounted-for water is estimated at greater than 30%. It is anticipated that this percentage figure can be reduced through the utilization of waste surveys, leak detection, and tighter construction inspection.

TABLE IV-4

CONSUMPTION DATA

YEAR	AVERAGE DOMESTIC PER CAPITA CONSUMPTION 1/c/d	INDUSTRIAL, COMMERCIAL AND INSTITUTIONAL % of domestic	1/c/d	UNACCOUNTED-FOR WATER 20% of [(2)+(4)] 1/c/d	PER CAPITA CONSUMPTION (2)+(4)+(5) 1/c/d
(1)	(2)	(3)	(4)	(5)	(6)
1972	49	10	5	11	65
1987	69	20	14	17	100
2000	86	25	22	22	130

Table IV-5 shows the estimated demands for the Supply Group I and II communities included in the UNDP(SF) project study area based on population estimates for the years 1987 and 2000, respectively.

It is noted in connection with this table that the population totals for some of the above communities are subject to change in the final design phase. This is because several of the communities overlap or, conversely, the interlying populated areas are included in other water supply systems.

DESIGN CRITERIA

Population Growth

As previously mentioned, the average growth in population was assumed at approximately 4% per year since actual trends or concentrations cannot be accurately prophesied. All water supply projects included in the master plan of this UNDP(SF) Project are based on this population increase with an interim design population reached after 15 years. Local industrial expansion is included as a percentage of the interim design figure.

Rate of Water Usage

The apportionment of the domestic demands in individual project areas was taken as proportional to the population served by each main, incorporating actual existing house counts whenever practicable. Population totals in such cases were calculated on the basis of six persons in each house.

The following rates of flow and variations thereof were used in designs:

Average daily usage = 100 liters per capita per day
Maximum daily flow : average daily flow = 1.40:1
Maximum hourly flow : average hourly flow = 2.50:1

The average daily flow of 100 l/c/d was used for design of the production elements of the water supply systems. The peak factor of 2.50:1 was used in the design of distribution system elements.

An investigation was conducted into the water requirements for specific users. The results of this study are listed in Table IV-6.

Distribution System

All systems were designed utilizing the Hazen and Williams formula:

TABLE IV-5

AVERAGE DAILY DOMESTIC WATER DEMANDS BY POPULATION

	Estimated Population		Estimated Daily Water Consumption	
	1987	2002	1987	2002
Supply Group I			(m³)	
A. SWC areas of supply:	+)			
Kwatta Leidingen	23,800		2,380	5,580
Koewarasan	6,000	10,300	600	1,400
Uitkijk and Jarikaba	10,300	17,800	1,030	2,400
Houttuin	7,100	12,200	710	1,640
Domburg	12,500	21,500	1,250	2,900
Meerzorg and Peperpot	15,000	26,000	1,500	3,500
Jagtlust, Lust and Rust, and Belwaarde	1,100	1,900	110	260
Nieuw Amsterdam and Voorburg	6,430	11,100	640	1,500
Marienburg	18,550	32,000	1,860	4,300
Alkmaar	4,640	8,000	460	1,080
Spieringshoek-Katwijk and Wederzorg	4,930	8,500	490	1,140
Tamanredjo	5,160	8,850	520	1,200
Leliendal	1,600	2,000	120	270
B. Surinam River Industrial Area:				
Paranam	3,800	6,550	380	690
Billiton	1,520	2,650	150	360
Smalkalden-La Vigilantia	2,160	3,750	220	380
Supply Group II				
A. Saramacca Distr.:				
Groningen and Tambaredjo	4,400	7,600	440	1,020
Calcutta and Tijgerkreek	7,050	12,200	700	1,650
Hildesheim	2,160	3,750	220	520
Kampong Baroe	4,330	7,500	430	1,020
B. Coronie Distr.:				
Totness	4,860	8,400	490	1,140
C. Nickerie Distr.:				
SWC area of supply	18,000	31,800	1,800	4,300
Wageningen	5,200	9,000	520	1,220
Groot en Klein Henar	6,300	10,900	630	1,470
Paradise	15,190	26,300	1,520	3,550
Corantijn and Van Drimmelen Polder	15,190	26,300	1,520	3,550
D. Brokopondo Distr.:				
Brokopondo	1,620	2,800	160	380
Klaaskreek	3,000	5,150	300	700
Brownsweg	3,800	6,550	380	880
Lombé and Mooie Kreek	2,000	3,450	200	470
E. Commewijne Distr./ Marowijne Distr.:				
Alliance	755	1,350	70	140
Constantia, Killenstein and l'Esperance	1,730	3,000	170	400
Moengo and Wonoredjo	10,800	18,800	1,080	2,550
SWC Area (Albina)	4,860	8,400	490	1,140

+) See Paramaribo study report.

TABLE IV-6

WATER DEMANDS FOR SPECIFIC PURPOSES (PEAK FLOWS)

Type of Consumer	Average Capacity (Max. Condition)	Peak Daily Usage (m^3)	Peak Hourly Usage (m^3)	Remarks
Cinemas	1,000 persons	3.0	0.5	Three performances, 2 hours each
Restaurants	35 tables x 4 persons	6.5	0.65	Max. trade between noon Saturday and 1:00 a.m. Sunday
Local shops	100 persons/hr.	1.0	0.2	Max. trade 4 hours daily; includes cleaning water
Schools	Varies	6.0	0.8	Flows based on investigations at 12 grade schols and four preschools
Service stations	Varies	21.0	1.3	Sixteen business hours per day; includes car washing
Swimming pools	900 m^3 volume	900.0	37.5	For monthly draining and cleaning - 24-hour period
		90.0	3.75	Average for 4-week period
Government offices	40 employees	2.0	0.3	Includes cleaning and gardening
Recreation buildings	200 persons	10.0	1.6	Max. usage during 6-hour period on Saturday
Medical buildings	50 visitors - 5 staff	1.5	0.2	Eight-hour day
Churches	Varies	0.5/hectare	-	Calculate on surface area basis
Equipment repair shops	Varies	7.0	0.9	Eight-hour day

$$Q = 0.0103\ C\ H^{0.54}\ D^{2.63}$$

Q = rate of flow, cfs.
C = friction coefficient
H = head loss per 1,000 ft. of straight pipe
D = diameter of pipe, ft.
For cast iron pipe, C = 120
For plastic pipe, C = 130
For A.C. pipe, C = 130

In Surinam, the use of plastic (polyvinylchloride) pipe with cemented joints and certified according to ASTM (USA) or KIWA (Netherlands) standards for water supply systems is customary for diameters through six inches. Above six inches asbestos-cement pipe of ISO (International) standard is used. For design purposes in this study, plastic and asbestos-cement pipes were specified for distribution systems except in those cases where conditions of installation warranted the need for cast-iron pipe.

Pressures in Distribution Systems

In order to analyze pressures at the junction points of the distribution systems, the Hardy Cross method of successive approximation was utilized. As an audit and also for demonstration purposes, computer analyses were employed on representative systems. This node balancing type of analysis was based on a modified Hardy Cross method. A detailed discussion is included in a later section of this report.

Since most homes and buildings in rural areas do not exceed two stories in height, a minimum residual pressure of 20 psi was maintained in the designs.

Fire Protection

After careful analysis of the local conditions in the rural or semi-urban areas which are typical for this UNDP(SF) Project, it was decided that no special fire-flow provisions need be considered, mainly because the dwellings are widely spaced and hydrants for fire protection could not be economically justified. Actually, most of the dwellings are located along irrigation canals with water readily available at all times in case of fire. However, if fire protection is justified at some time in the future in the built-up areas, minimum pressures of 20 psi are available at the fire hydrant locations. Assumptions are made that these areas would be served by pumping engines and that a fire station would be centrally located to serve the community.

Rate of Flow in the Distribution Systems

The systems were designed for peak flows, 2.5 x average flow. The number of users was arrived at by adding the present population and, depending on the area, the estimated future population. To this total were added the miscellaneous flows, such as industrial, commercial, and institutional.

Storage

A ground storage tank containing 50% of the total average daily flows was designed at the supply point. In addition to this reservoir, elevated storage with a volume necessary to provide adequate working pressures in the distribution system was also included in the individual designs.

Pumping Stations

Small installations were designed for the average daily flow. The pumps would operate on two eight-hour shifts, since automation of the pumping station was kept to a minimum. The designs included space for a small office, storage space, transformer house, and living space for the caretaker (only if the size of the station warranted his full-time presence). Electricity was used as the main supply of power. Diesel engines were included to provide standby power for the larger stations.

Pipeline Corrosion and Incrustation

It was expected that neither corrosion nor incrustation would constitute a problem since the pipe to be used would be plastic, asbestos-cement, or cast-iron, and Surinam water does not affect these materials adversely.

During the early stages of the project, samples of plastic pipe were taken from a six-year old water supply system in order to examine these for deposition, incrustation, or deformation. While there appeared to be a very small amount of iron deposited, no serious problems could be attributed to any of the above sources of difficulty.

Design Periods

(a) Land Requirements

The present situation in Surinam covering land ownership is such that, except in densely populated areas, most of the land is under central government control. Therefore, the procurement of land for major elements in the water supply systems should pose no problem even if these needs are projected over a 40-year period. It was assumed, however, that procurement would proceed within a reasonable length of time, thus precluding the occurrence of later problems.

(b) Major Production Elements

The design period for buildings as well as the concrete structures such as filters and storage was estimated at 40 years. The mechanical equipment design period was 15 years. Elevated storage design period was estimated at 25 years. Electrical wiring, controls, and so on were designed for a period of 10 years. The design period of wells was estimated at seven years, when replacement would become necessary.

(c) Distribution Elements

Because the pipe sizes were relatively small, and the unit costs were proportionally not great, the financial considerations were such that the design period could be estimated at 40 years. Similarly, the difference in construction costs between short- and long-term design periods tended to be moderate because of the small sizes involved.

CONSTRUCTION CONSIDERATIONS

Construction Standards

Generally, the standards of quality of construction for proposed water supplies were similar to those already in effect in the country, since extremely divergent applications would not be justified. In the case of concrete construction, for example, it is understood that the actual strengths achieved in concrete produced in Surinam tend to be somewhat less than those of other countries. This is reflected in the structural designs of the filter plant units.

The unavailability in Surinam of European standards for fittings and pumps for design purposes at the time the Project was initiated created the necessity for establishing the U.S. standards for these elements of the water supply projects. For piping, the ASTM and the KIWA standards were adopted. Thus, for construction purposes the equipment and materials specified according to the U.S. standards could be substituted with acceptable European material. The designs are such that any minor differences in dimensions can be compensated for with few problems.

Construction Labor, Materials, and Equipment

The availability of skilled and unskilled labor in the country is such that the construction of the water supply projects should proceed without too much difficulty. A number of projects of considerable size and complexity have been completed in Surinam in which local labor took an active part. In connection with the bauxite industry particularly, local labor was used extensively in the construction of plant facilities, roads and bridges, dams, and water installations. Also, the operation of heavy-duty construction equipment was initiated and developed to a great degree. The Surinam Water Company built several systems in which local labor had a large part. The "trainability" of local labor is entirely feasible and has been demonstrated many times.

Generally speaking, the technical and supervisory personnel available in the country are already committed to existing organizations.

Private and government sector organizations compete for services on this level of employees, and therefore the number of technical, supervisory, and professional personnel is limited. However, the staff that is available seems to be competent and capable of handling the responsibilities connected with water supply projects.

Cement, aggregates, water for concrete, and lumber are readily obtainable in the country and are in stock. Items such as reinforcing steel, structural steel, hardware, pumps, motors, electrical wiring and switchgear, glass, paint, and piping must be imported and sufficient lead time must be allowed for this importation.

Logistical items such as housing, food, fuel and lubricants, repair facilities, and the like are not likely to pose any serious problems.

Heavy-duty construction equipment and specialized construction equipment is available from several contractors in the metropolitan areas. The maintenance and repair facilities for these are also present.

Subcontracting of some of the branches of the work such as electrical and mechanical could be possible since there are contractors of this type actively engaged in these fields at present. However, the individual subcontracts might prove to be somewhat expensive since the amount of this specialized work involved on the project is not very great.

Effect of Weather

Construction may be affected by the rainy seasons which normally occur twice during the year. The seasons and rainfall can be shown as follows:

Month	J	F	M	A	M	J	J	A	S	O	N	D	Year Total
Season	Rainy		Short dry		Long rainy				Long dry			Rainy	
Average Rainfall	193	150	162	232	321	303	226	167	86	87	109	174	2,208 mm

Some delays due to seasonal rains might be anticipated. Although the rains may be heavy, they are of short duration; and the normal practice in construction during the rainy seasons seems to be to work in spite of the rain. Temperatures tend to remain fairly constant, with humidity contributing to personal discomfort more than does excessive temperature. However, it is expected that no serious construction delays would stem from existing precipitation, temperatures, or weather conditions and that the overall effect on the construction schedules, equipment use, and labor forces would be minimal.

CHAPTER 3

WATER QUALITY REQUIREMENTS

GENERAL

This chapter deals with the development of the future water demands of the project area, principally as affected by the water quality requirements for drinking water supplies. Water demands, as used herein, refer to diversion requirements and not to consumptive use, or in other words, the established water demands represent the amounts of water which have to be derived from either ground water or surface sources.

The water quality standards presented represent the minimum requirements for bacteriological, chemical, and physical characteristics of water for domestic use. These standards will be helpful in assessing the suitability of the various sources of water supply and in determining the type and degree of treatment to be given.

QUANTITY REQUIREMENTS

The total water requirement of the project areas is that quantity which must be available to meet the estimated future domestic and industrial water demands. Detailed discussions of the determination of the basic domestic and industrial requirements were presented in an earlier section.

QUALITY STANDARDS

Standards relating to the quality of public water supply systems are set forth in the World Health Organization's International Drinking Water Standards, 1971 edition. These standards prescribe minimum requirements for the bacteriological, virological, biological, radiological, and physical, chemical, and esthetic characteristics of drinking water supplies. The WHO International Drinking Water Standards are considered to be attainable by all countries throughout the world at the present time. The WHO European Standards for Drinking Water (1970) were also reviewed for possible application to the Surinam water supplies, but it was decided to consider these as a "next step" in development.

It should be noted that the quality requirements discussed herein are those which, unless stated otherwise, must be maintained within the distribution system or, in other words, reflect water qualities at the point of use. These standards serve as a very useful tool in assessing the relative suitability of various sources of supply, and as such should not be regarded as implying approval of the degradation of an existing source which might be of superior quality to that recommended.

BACTERIOLOGICAL CHARACTERISTICS

The use of a new source for water supply requires the full bacteriological examination of the source. Recent contamination by human excrement or animal pollution constitutes the greatest danger. However, pathogenic bacteria cannot be isolated in a practical manner during routine testing; and, therefore, the presence of E. coli and the coliform group as a whole in samples is commonly interpreted as indication of pollution.

For piped water supplies, the following standards obtain:

Water Entering the Distribution Systems

(1) Chlorinated or otherwise disinfected supplies:

No coliform organisms in any 100 ml sample.

(2) Non-disinfected supplies:

No *E. coli* in any 100 ml sample.
Not more than three coliform organisms per 100 ml (occasionally).

Water in the Distribution System

(1) Throughout any year, 95% of samples should not contain any coliform organisms in 100 m.

(2) No sample should contain *E. coli* in 100 ml.

(3) No sample should contain more than 10 coliform organisms per 100 ml.

(4) Coliform organisms should not be detectable in 100 ml of any two consecutive samples.

For small individual or community supplies the following standard is recommended:

The coliform count should be less than 10 per 100 ml.

PHYSICAL, CHEMICAL, AND ESTHETIC CHARACTERISTICS

Table IV-7 refers to the acceptability of water for domestic use. Examinations for some characteristics should be carried out routinely, with others conducted less frequently. Where a new source is considered, all the tests should be performed.

TABLE IV-7

SUBSTANCES AND CHARACTERISTICS AFFECTING THE ACCEPTABILITY OF WATER FOR DOMESTIC USE

Substance or Characteristic	Undesirable Effect that May be Produced	Highest Desirable Level	Maximum Permissible Level
Substances causing discoloration	Discoloration	5 units[a]	50 units[a]
Substances causing odors	Odors	Unobjectionable	Unobjectionable
Substances causing taste	Tastes	Unobjectionable	Unobjectionable
Suspended matter	Turbidity Possibly gastrointestinal irritation	5 units[b]	25 units[b]
Total solids	Taste Gastrointestinal irritation	500 mg/1	1,500 mg/1
pH range	Taste Corrosion	7.0 to 8.5	6.5 to 9.2
Anionic detergents[c]	Taste and foaming	0.2 mg/1	1.0 mg/1
Mineral oil	Taste and odor after	0.01 mg/1	0.30 mg/1

[a] On the platinum-cobalt scale

[b] Turbidity units

[c] Different reference substances are used in different countries

TABLE IV-7 (cont.)

SUBSTANCES AND CHARACTERISTICS AFFECTING THE ACCEPTABILITY OF WATER FOR DOMESTIC USE

Substance or Characteristic	Undesirable Effect that May be Produced	Highest Desirable Level	Maximum Permissible Level
Phenolic compounds (as phenol)	Taste, particularly in chlorinated water	0.001 mg/1	0.002 mg/1
Total hardness	Excessive scale formation	2mEq/[a, b] (100 mg/1 $CaCO_3$)	10mEq/1 (500 mg/1 $CaCO_3$)
Calcium (as Ca)	Excessive scale formation	75 mg/1	200 mg/1
Chloride (as Cl)	Taste; corrosion in hot-water systems	200 mg/1	600 mg/1
Copper (as Cu)	Astringent taste; discoloration and corrosion of pipes, fittings, and utensils	0.05 mg/1	1.5 mg/1
Iron (total as Fe)	Taste; discoloration; deposits and growth of iron bacteria; turbidity	0.1 mg/1	1.0 mg/1
Magnesium (as Mg)	Hardness; taste; gastrointestinal irritation in the presence of sulfate	Not more than 30 mg/1 if there are 250 mg/1 of sulfate; if there is less sulfate, magnesium up to 150 mg/1 may be allowed	150 mg/1
Manganese (as Mn)	Taste; discoloration; deposits in pipes; turbidity	0.05 mg/1	0.5 mg/1

[a] If the hardness is much less than this, other undesirable effects may be caused; for example, heavy metals may be dissolved out of pipes.

[b] ImEq/1 of hardness-producing ion = 50 mg $CaCO_3$l = 5.0 French degrees of hardness = 2.8 (approx.) German degrees of hardness = 3.5 (approx.) English degrees of hardness.

TABLE IV-7 (cont.)

SUBSTANCES AND CHARACTERISTICS AFFECTING THE ACCEPTABILITY OF WATER FOR DOMESTIC USE

Substance or Characteristic	Undesirable Effect that May be Produced	Highest Desirable Level	Maximum Permissible Level
Sulfate (as So_4)	Gastrointestinal irritation when magnesium or sodium are present	200 mg/l	400 mg/l
Zinc (as Zn)	Astringent taste; opalescence and sand-like deposits	5.0 mg/l	15 mg/l

Industries are generally willing to accept for most processes water that meets drinking water standards. Where water of higher quality is needed, for example, for certain electronic equipment manufacture, food and beverage preparation, or for high-pressure boilers, industry must recognize that additional water treatment is the responsibility of the water user.

Some chemicals, when present in certain concentrations in drinking water, are considered to constitute a hazard to health. The International Standards for Drinking Water include limits for these, based on a daily intake of 2.5 liters for a man weighing 70 kg. These are shown in Table IV-8.

Although contamination of drinking water with pesticides, insecticides, herbicides, and fungicides can reach the level of constituting a toxicity hazard, the influence of such contamination on water supplies in Surinam has been limited, because the sources have generally been ground water sources.

While the following chemical substances do not, as yet, constitute problems in the water supply systems of Surinam, it is deemed worthwhile to direct attention toward them since this detection (if necessary) serves to protect the health of the users of drinking water.

Fluoride concentration in water supplies must be assessed in relation to the individual's total daily fluoride intake, particularly since fluoride-containing foods form part of the diet. The concentrations of fluorides recommended for drinking water for various temperature ranges (five-year average daily temperatures) are listed in Table IV-9.

Certain carcinogenic substances in water supplies are considered as constituting health hazards. Among these are nitrosamines and some polynuclear aromatic hydrocarbons (PAH). Nitrates in concentration greater than 45 mg/1 (expressed as NO_3) can be detrimental to children, especially infants, after the nitrates have been reduced to nitrites and these, after ingestion, produce nitrosamines. In the case of PAH, the concentration of six PAH compounds should not in general exceed 0.2 mg/1.

VIROLOGICAL CHARACTERISTICS

Although viruses can sometimes be found in raw water, the usual transmission occurs through water containing coliform organisms frequently in reclaimed or reused water which is untreated sewage effluent. Sedimentation and slow filtration contribute to virus removal. Inactivation can be achieved by oxidation utilizing a concentration of 0.5 mg/1 of free chlorine for one hour.

TABLE IV-8

TENTATIVE LIMITS FOR TOXIC SUBSTANCES IN DRINKING WATER

Substance	Upper Limit of Concentration
Arsenic (as As)	0.05 mg/1
Cadmium (as Cd)	0.01 mg/1
Cyanide (as CN)	0.05 mg/1
Lead (as Pb)	0.1 mg/1
Mercury (total as Hg)	0.001 mg/1
Selenium (as Se)	0.01 mg/1

TABLE IV-9

RECOMMENDED CONTROL LIMITS FOR FLUORIDES IN DRINKING WATER

Annual Average of Maximum Daily Air Temperature in ^{o}C	Recommended Control Limits for Fluorides (as F) in mg/1	
	Lower	Upper
10-12	0.9	1.7
12.1 - 14.6	0.8	1.5
14.7 - 17.6	0.8	1.3
17.7 - 21.4	0.7	1.2
21.5 - 26.2	0.7	1.0
26.3 - 32.6	0.6	0.8

WHO International Standards for Drinking Water, 1968.

BIOLOGICAL CHARACTERISTICS

Biological qualities of drinking water in Surinam are of main importance in the effects of such characteristics in tastes and odors of water. Generally, biological growths such as slimes, algae, and animalcules can be controlled (if required) through chlorination, particularly in those treatment plants which do not incorporate conventional flocculation and filtration processes.

RADIOLOGICAL CHARACTERISTICS

Local conditions in Surinam tend to preclude significant radioactive contamination of water supplies. However, the levels of radioactivity recommended by the International Commission on Radiological Protection (ICRP) are listed below for informational purposes:

Gross alpha activity 3pCi/l
Gross beta activity 30pCi/l

These levels apply to the mean of all radioactivity measurements during a three-month period.

CHAPTER 4

SOURCES OF SUPPLY

GENERAL

Concurrent with the determination of the magnitude of existing and anticipated demands for potable water, investigations were undertaken to locate suitable sources. A general discussion of the occurrence of water in Surinam follows.

Surinam receives an apparent abundance of rainfall, most of which falls during the periods April to July and December to February. A large part of this precipitation is carried northward to the sea by the several rivers and their tributaries; the rest, except for losses by evaporation and transpiration, seeps into the earth and may serve to recharge ground water supplies. Unfortunately, surface supplies from streams, from a quality standpoint, are not always suitable for domestic consumption, unless needed conditioning is accomplished through suitable water treatment.

In most areas some rain water is available from catchments, and this is a popular source of water during the wet seasons. However, catchment areas and collection tanks have only limited capacities and supplies are generally depleted well before the rains of the next wet season commence. Thus, during the hot, dry seasons when water is most urgently needed, rain water supplies become exhausted and acute problems develop; and this requires emergency action by government officials, which involves the previously discussed transporting of water over great distances for distribution to the people. These situations show that rain water catchment in the vast majority of cases cannot be depended upon to supply reliable water service throughout the year.

Ground water supplies also present problems; this is true largely from a standpoint of chemical quality. Water from aquifers in the coastal plain, where the larger concentrations of population are found, frequently contains large amounts of chlorides. Further inland, water of lower chloride content may be found in many places but generally it has other objectionable properties due to the presence of iron salts and gases such as hydrogen sulfide, carbon dioxide, and methane. This problem is usually more pronounced in water from the deeper strata. Simple treatment methods can be employed (essentially aeration, pH adjustment, rapid sand filtration, and shell filtration) to effect removal of iron and objectionable gases, but a treatment plant with suitable equipment is required. The treatment process also serves to stabilize water which has a tendency to be corrosive.

In 1956, SABTS initiated a project for demonstrating practical methods of installing low-yield, shallow, sanitary wells for supplying water for farms and small groups of families. On the basis of 230 wells installed, it was concluded that it is possible in many places in Surinam to develop sanitary wells, the water from which does not contain excessive amounts of iron. Also, it had been observed that some fresh water is generally found in the upper layers of the aquifers, indicating that low-yield wells might be developed to supply water needs of small groups without having to resort to the installation of elaborate treatment facilities. The SABTS demonstration developed valuable information about the ground water supplies, but considerably more exploratory work was needed in all of the districts in order to gain a more comprehensive view of the possibilities for solving the basic problems.

A detailed evaluation of the available and potential sources of surface and ground water is provided in this report in Volume III - Water Resources (Hydrogeological and Hydrological Studies). The water balance method was selected for source evaluation purposes with emphasis placed upon hydrologic cycle studies. The following elements were included: rainfall, evaporation, evapotranspiration, infiltration and percolation, ground water flow, and stream flow.

RAIN WATER

In its natural state, rain water is almost completely void of chemicals and if collected on clean catchment surfaces may be relatively free of contamination. However, it is difficult from a practical standpoint to maintain good sanitary conditions in catchments and collecting tanks; also, handling methods employed are apt to cause contamination of the water.

Holding of rain water for long periods of time (without replenishing), especially during abnormally long dry seasons, requires extensive storage capacities, making provision of adequate facilities impractical for most communities.

Further, in Surinam rain water collection does not constitute a dependable method of water supply because rainy seasons do not always provide the total precipitation required. Precipitation information gathered over 30 years shows that the west-central coastal area is rather dry, with more precipitation towards the east, the extreme west, and in the higher interior. Comparison of the 30-year information with the 1961-1966 data shows that the latter are about 20% (400-500 mm) lower than the figures from the 30-year data. It is not known if this dry period extended farther into the interior, but it seems reasonable to assume that the mean values for the 1961-1966 period were below normal (see Figure IV-1).

In a precipitation study covering several Surinam and foreign areas conducted in connection with a river and coastal transportation project, it was determined that the most extreme rainfall patterns occur at Georgetown and Cayenne, which have two and one distinct rainy seasons respectively. In Surinam the seasonal variations showed a transition between these two, with a recurring rainy season during April, May, and June, but with a variable intensity of the short rainy period in December and January. It was characteristic of the southwest part of the country that one long dry season lasting about eight months occurred annually. A similar, agriculturally disastrous situation seems to occur in the coastal plain when, in approximate 10-year cycles, rains fail to materialize in the short dry season.

SURFACE WATER

Water from surface supplies comprising rivers, swamps, lakes, and canals, including those in the more remote, undeveloped sections of the country, can be used for domestic consumption, but requires suitable treatment. High quality water which is satisfactory both chemically and bacteriologically can be produced in treatment plants at a relatively nominal cost, provided the system is of such magnitude as to justify the cost of treatment plant installation.

The principal occurrence of surface water in Surinam is in the many rivers and streams which traverse the country. A considerable amount of hydrological and meteorological investigation has been done and is continuing. The Hydraulic Research Division of the Ministry of Public Works conducts a continuing program in the study of the quality and availability of river water in Surinam. Hydrological data reports covering the lower Corantijn River, the Nickerie River, and the lower Marowijne River have been published; and another report which concerns the lower Suriname River is ready for release.

Correlation of river discharges and comparison of precipitation data with discharges have been attempted by the Hydraulic Research Division in order to estimate flows in the main Surinam rivers. One estimate of the mean flow at river mouth is shown as follows:

River	Discharge m^3 per sec.	km^3 per Annum	Run-off mm per Annum
Corantijn	2,000	63	950
Nickerie	200	6.3	650
Coppename	470	15	810
Saramacca	240	7.5	735
Suriname	440	14	800
Commewijne	120	3.8	565
Marowijne	2,000	63	950

Variations in the discharges of Surinam rivers are caused by several factors. The hydrographs of the Corantijn and Coppename Rivers show strong variations in consonance with the periods of heavy precipitation. The regimen of the lower Suriname River is completely governed by the operation of the hydroelectric power station in the dam near Afobaka, which started operating in October 1965 following a period of complete cessation of flow during 1964.

The sediment discharges of the rivers are relatively small. This, together with strong tides, causes the estuaries to reflect considerable wear.

The tide along the Surinam coast is diurnal, and the mean tidal range is approximately two meters at the river mouths. Penetration of the tidal wave is deep, and during periods of low flow the tidal influence can in some cases be detected over 200 km inland from the river mouth.

A continual interaction takes place between the more dense ocean water moving upstream along the estuary bottom, reinforced by the tidal currents, and the river discharge which tends to push the "salt-wedge" back into the ocean. River discharge, therefore, tends to control the movement of the salt-wedge. In the case of the Suriname River, for example, the so-called "acceptable" level of chlorides of 100 mg/l extends to Torarica (105 km) during periods of minimal discharge.

Several proposals are being considered by the Government which may have a decided effect on river discharges and their control. Principally, these are hydroelectric projects which would be associated with the bauxite exploitation and aluminum production.

The initial hydroelectric station, with a capacity of 180 MW, was constructed at Afobaka, approximately 150 kilometers upstream from Paramaribo on the Suriname River. Completed in early 1964, the dam impounds a water storage covering an area of about 1,600 square kilometers. Controlled discharge at Brokopondo provides a mean yearly flow of approximately 250 m^3/sec. A preliminary study and plans have been developed for another dam on the Suriname River. Located at Joden Savanna, the Torarica Dam hydroelectric plant would deliver 18 MW.

The Ministry of Development submitted a request to the United Nations Development Programme for a feasibility study covering the Kabalebo Hydroelectric Scheme in which the Kabalebo River would be dammed at Avanavero Falls and Kabalebo Falls, linking the Corantijn River at Frederik Willem IV Falls with the upper Kabalebo River. Through the diversion of the Corantijn River, hydroelectric power of up to 870 MW could be developed in an average year.

Additional diversion of the Corantijn into the Nanni River reservoir has also been studied from the point of view of providing water for the Wageningen rice polders as needed.

Another plan, for a dam at Stondansie on the Nickerie River, was concerned with the regulation of the lower Nickerie River for irrigation purposes.

The possibilities of developing hydroelectric power on the Lawa, Tapanahony, and Marowijne Rivers have been initially investigated with several potential dam sites being considered. The Lawa and Marowijne Rivers, of course, are also of interest to the French Government.

A map, prepared by the Hydraulic Research Division of the Ministry of Public Works, showing the pertinent discharge measurement stations and other stream data collection points for the lower Suriname River, is included as Figure IV-2.

A topographical map with longitudinal profile of the lower Suriname River is provided as Figure IV-3.

Investigation covering the quality of river waters, particularly in the estuaries, was initiated in 1960 by the Bureau for Rural Development, and responsibility for this operation was assumed by the Hydraulics Research Division in 1964. So-called "salt-trips" are part of this investigation, and 88 sampling tours were conducted from 1960 to 1970 in order to determine the surface chlorides at given points in the river.

While the ground water sources appeared to be adequate for community water supply demands throughout the master planning period, it was deemed necessary to consider the alternative of providing surface water sources. Thus, early consideration was given in the project to the possibility of utilizing surface water. Among other approaches, research and development of the biological or slow-sand filter was instituted, and a test program was initiated in several areas in which it might have been feasible to use surface water.

Inherent problems exist in some areas, and these either preclude or severely restrict the utilization of surface water.

The most obvious source of large quantities of surface water is, of course, the van Blommensteinmeer, the large man-made lake which lies approximately 120 km south of Paramaribo and supplies water to the hydroelectric facility at Afobaka Dam. While it appears a not-too-difficult task to design a transmission line from this lake to serve the populated areas to the north (including Paramaribo), there are several impelling reasons why this had not been accomplished in the past. First, and most important, the Brokopondo Agreement between the Government of Surinam and the aluminum company stipulates that all the water impounded behind the dam is under the control of the company and is not available for exploitation. Next, even should the water become available for community supplies, the present needs are being met closer to the populated coastal area, thereby obviating the need for a large present investment in

CHAPTER 6

INTERIM WATER SUPPLIES

STATUS OF EXISTING WATER SUPPLIES

As of beginning 1972 the following status existed with regard to community water supply systems in Surinam:

Existing Operational Systems

1. Paramaribo, Albina, and Nieuw Nickerie (Surinam Water Company)
2. Koewarasan (Ministry of Rural Government and Decentralization)
3. Meerzorg (Surinam Water Company)
4. Groningen-Tambaredjo (Surinam Water Company)
5. Brokopondo (Surinam Water Company)
6. Klaaskreek (Surinam Water Company)
7. Brownsweg (Surinam Water Company)
8. Alliance (Ministry of Agriculture)
9. Totness (Ministry of Agriculture)
10. Kampong Baroe (Ministry of Agriculture)
11. Groot Chatillon (Ministry of Health)
12. Paranam (Suralco)
13. Moengo (Suralco)
14. Afobaka (Suralco)
15. Onverdacht (Billiton Mij.)
16. Smalkalden (Billiton Mij.)
17. Mariënburg (Cultuur and Rubber Mij.)
18. Wageningen (SML)
19. Wonoredjo (Ministry

Systems under Expansion

1. Paramaribo (Surinam Water Company)
2. Groningen
3. Brokopondo
4. Koewarasan

Systems under Planning

1. Brokopondo (expansion)
2. Coronie (elevated storage)
3. Meerzorg and Peperpot (expansion)
4. Kwatta-Leidengen
5. Kwakoegron
6. Groot en Klein Henar
7. Corantijn and van Drimmelen Polder
8. Paradise
9. Calcutta-Tijgerkreek West
10. Nieuw Lombe-Mooie Kreek
11. Pad van Wanica West
12. Uitkijk-Jarikaba
13. Jagtlust-Nw. Amsterdam-Voorberg, Alkmaar Zoelen, Mariënburg
14. Spieringshoek-Wederzorg and Katwyk
15. Tamanredjo
16. Leliendal

Another plan, for a dam at Stondansie on the Nickerie River, was concerned with the regulation of the lower Nickerie River for irrigation purposes.

The possibilities of developing hydroelectric power on the Lawa, Tapanahony, and Marowijne Rivers have been initially investigated with several potential dam sites being considered. The Lawa and Marowijne Rivers, of course, are also of interest to the French Government.

A map, prepared by the Hydraulic Research Division of the Ministry of Public Works, showing the pertinent discharge measurement stations and other stream data collection points for the lower Suriname River, is included as Figure IV-2.

A topographical map with longitudinal profile of the lower Suriname River is provided as Figure IV-3.

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Inherent problems exist in some areas, and these either preclude or severely restrict the utilization of surface water.

The most obvious source of large quantities of surface water is, of course, the van Blommensteinmeer, the large man-made lake which lies approximately 120 km south of Paramaribo and supplies water to the hydroelectric facility at Afobaka Dam. While it appears a not-too-difficult task to design a transmission line from this lake to serve the populated areas to the north (including Paramaribo), there are several impelling reasons why this had not been accomplished in the past. First, and most important, the Brokopondo Agreement between the Government of Surinam and the aluminum company stipulates that all the water impounded behind the dam is under the control of the company and is not available for exploitation. Next, even should the water become available for community supplies, the present needs are being met closer to the populated coastal area, thereby obviating the need for a large present investment in treatment and transmission facilities. Finally, the level in the lake has not retained the height considered more than sufficient for maximum power production, and any large extraction for uses other than hydroelectric power generation would not be justifiable.

Also considered were the following: Suriname River, Para Creek, Surnau Creek, Saramacca River, and Orleana Creek. All were studied in the context of providing raw water for the lower Suriname basin area projects. Although each showed promise, the frequency of contamination occurrences through high chloride levels in each of these surface water sources caused them to be rejected. A six-month chemical sampling program was conducted, however. In the case of the Suriname and the Saramacca Rivers it is possible to install intakes, but the distances from the coastal area require relatively long transmission mains. Specific utilization of surface supplies is reported in greater detail in later sections of this report.

SWAMP SUPPLIES

The possibility of developing reservoirs of surface water in swampy areas was also given primary consideration, particularly in areas such as Commewijne where suitable ground water seemed to be virtually nonexistent, but this possibility was rejected for several reasons.

Some experience had been gained in impounding swamp water in connection with the private supply at Mariënburg serving approximately 4,000 persons, and it was found that aquatic weed growth and algae growth presented serious problems, as did chlorination.

In order to store sufficient quantities of water to provide a continuous supply over the dry seasons, it would be necessary to construct large reservoirs. For example, to serve 10,000 persons at an average daily demand of 100 l/c/d for a three-month dry season would require a reservoir containing 90,000 m^3 disregarding evaporation. At a depth of 2 meters, a net area of 45,000 sq. meters would be required. Construction of an impervious reservoir of this size would be costly and the capital cost would also necessarily have to reflect pumping costs, since gravity flow would be impossible.

Additional treatment beyond the oxidation realized in storage would also be needed because the swamp water tends to contain considerable organic matter and other pollutants.

GROUND WATER

General and specific aspects of ground water as supply are considered in detail in Volume III of this report.

In connection with ground water, the possibility of utilizing windmills was reviewed as a low-cost method of supplying water. The conditions for proper operation are set out in a WHO monograph covering water supply for rural areas and small communities. Of the five conditions listed, the most important requirement is for winds of more than 8 km per hour at least 60% of the time. Because the winds in Surinam are generally weak (approximately Beaufort 1.2^{o}) and furthermore are not consistent, this approach was not pursued. It is understood that provision for pumping with hand- or animal-power could be substituted for wind energy, but this type of operation is not considered practical in Surinam.

WATER ANALYSIS

Collection and analysis of water samples were performed for both ground and surface water sources. The facilities of the Central Laboratory of the Public Health Department were utilized for chemical and bacteriological analyses. References to ground water analysis are provided in detail in Volume III of this report.

With regard to surface water sources, chemical sampling was conducted for each potential source, particularly in connection with the biological filtration test program. Typical results are included as Table IV-10.

In addition to the bacteriological testing performed by the Central Laboratory, field testing was accomplished employing membrane filter techniques. These methods and chemical testing with field kits were introduced as part of the on-the-job training of the national staff.

TABLE IV-10

FILTERED SURFACE WATER
SOURCE: PROF. DR. IR. W. J. V. BLOOMENSTEIN LAKE

CHEMICAL ANALYSIS

Substance or Property	Dry Season 1970					Rainy Season 1970	Dry Season 1970	
Ammonia NH_3	neg.	neg.	neg.				neg.	
Iron Fe	2.1	1.4	1.1				0.9	
Manganese Mn	neg.	neg.	neg.				neg.	
Residue	90	45	95				160	
Chloride Cl	6	8	13				19	
Nitrite NO_2	pos.	neg.	neg.				neg.	
Nitrate NO_3	pos.	neg.	neg.				neg.	
Sulfate SO_4	neg.	neg.	T.				T.	
pH	6.0	7.3	7.1				6.5	
Color	CL.	CL.	CL.				CL.	
Bic. hardness	0.8°	0.4°	0.6°				0.6°	
Tot. hardness	0.8°	0.8°	0.6°				0.6°	
$KM_n\ O_4$	30	27	45				25	
H_2S	neg.							
Bacteriological Examination								
Plate counts	>30							
Most probable number	0							
Eichmann	neg.							

1) * Note: All samples were light yellow.
No color scale was used.
CL = Clear, CD = Cloudy, TU = Turbid

TABLE IV-10

SURFACE WATER
SOURCE: PARARIVER AT HIGH WAY

CHEMICAL ANALYSIS

Substance or Property	Dry Season 1970			Rainy Season 1970					Dry Season 1970				
NH_3 Ammonia	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Iron Fe	3.8	2.4	3.5	2.4	3.7	1.2	2.0	1.7	2.7	2.	1.4		
Manganese Mn	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Residue	400	374	231	104	121	127	676	51	57	23	330		
Chloride Cl	117	104	40	21	16	42	49	35	27	11	151		
Nitrite NO_2	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Nitrate NO_3	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Sulfate SO_4	32	41	16	T.	T.	T.	T.	T.	T.	T.	29		
pH	6.1	5.7	5.9	5.8	7.0	6.3	7.0	5.9	6.1	6.0	6.7		
Color	CL.	CL.	TU.	TU.	CL.	CL.	TU.	CL.	CL.	CL.	CL.		
Bic. hardness D.H.	0.5^o	0.5^o	0.7^o	0.7^o	0.9^o	1.0^o	0.7^o	0.6^o	0.5^o	0.6^o	0.6^o		
Tot. hardness D.H.	3.6^o	3.6^o	1.6^o	1.0^o	1.0^o	1.4^o	1.1^o	0.6^o	1.1^o	0.7^o	3.7^o		
KM_n O_4 demand	25	50	94	67	83	85	65	46		50	34		
H_2S	neg.						neg.						
					Bacteriological Examination								
Plate counts	>300	>300	>300	900	190	900							
Most probable/single number		0	0	40.000	1,000	100							
Eichmann/test	pos.	0	0	pos.	pos.	pos.							

1) * Note: All samples were light yellow.
No color scale was used.
CL = Clear, CD = Cloudy, TU = Turbid.

TABLE IV-10

SURFACE WATER
SOURCE: SARAMACCA DOORSTEEK

CHEMICAL ANALYSIS

Substance or Property	Dry Season 1970			Rainy Season 1970				Dry Season 1970	
NH_3 Ammonia	neg.	neg.	neg.	pos.	neg.	neg.	neg.	neg.	neg.
Iron Fe	5.9	0.5	3.2	1.3	5.9	4.0	4.4	1.8	4.2
Manganese Mn	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	0.08
Residue	510	732	340	362	549	609	167	142	223
Chloride Cl	92	55	30	55	31	T.	22	35	65
Nitrite NO_2	neg.	neg.	neg.	Str. pos.	neg.	neg.	neg.	neg.	neg.
Nitrate NO_3	neg.	neg.	neg.	Str. pos.	neg.	neg.	neg.	neg.	neg.
Sulfate SO_4	29	92	5	19	neg.	T.	T.	T.	23
pH	7.1	7.4	7.6	6.8	7.5	6.7	6.8	6.8	8.1
Color	CL.	TU.	TU.	TU.	---	TU.	---	---	---
Bic. hardness D.H.	2.3^o	3.1^o	2.9^o	6.8^o	2.9^o	3.1^o	0.3^o	2.5^o	3.6^o
Tot. hardness D.H.	5.3^o	6.1^o	3.0^o	6.8^o	3.7^o	4.0^o	0.9^o	2.8^o	3.9^o
KM_n O_4 demand	39	55	55	59	59	59	47	39	40
H_2S	neg.				neg.	neg.	neg.		
				Bacteriological Examination					
				NIL					

1) * Note: All samples were light yellow.
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CL = Clear, CD = Cloudy, TU = Turbid.

TABLE IV-10

SURFACE WATER
SOURCE: PARA DOORSTEEK

CHEMICAL ANALYSIS

Substance or Property	Dry Season 1970			Rainy Season 1970					Dry Season 1970			
Ammonia NH_3	neg.	pos.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.
Iron Fe	1.2	1.6	1.1	2.4	2.3	1.3	1.1	1.4	1.3	1.0	0.45	2.1
Manganese Mn	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.
Residue	60	120	152	215	82		85			55	67	71
Chloride Cl	4	2	13	7	7	14	19	16	20	21	18	18
Nitrite NO_2	pos.	neg.	neg.	neg.	pos.	neg.	neg.	neg.	neg.	neg.	neg.	neg.
Nitrate NO_3	pos.	pos.	neg.	neg.	pos.	neg.	neg.	neg.	neg.	neg.	neg.	neg.
Sulfate SO_4	neg.	neg.	10	TU.	TU.	neg.	TU.	TU.	TU.	TU.	neg.	neg.
pH	3.6	6.4	5.0	4.5	7.3	4.3	6.2	5.9	5.6	6.2	6.4	6.4
Color	CL.	CL.	TU.	TU.	CL.	CL.	TU.	CL.	CL.	CL.	CL.	CL.
Bic. hardness D.H.	0.5°	1.0°	0.7°	0.4°	1.0°	0.5°	0.1°	0.5°	0.3°	1.0°	0.3°	
Tot. hardness D.H.	3.6°	1.0°	0.7°	0.6°	1.1°	0.5°	0.4°	0.6°	0.3°	1.0°	0.3°	
$KM_n\ O_4$ demand	25	31	60	85	80	92	78	37	78	42	79	
H_2S	neg.						neg.	neg.				
	Bacteriological Examination											
Plate counts	71	4	>300	>330	30	210						
Most probable/single number		0	0	5,000	8,000	100						
Eighmann/test	pos.	0	0	pos.	pos.	pos.						

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No color scale was used.
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TABLE IV-10

SURFACE WATER
SOURCE: TOUT LUI FAUT KANAAL

CHEMICAL ANALYSIS

Substance or Property	Dry Season 1970			Rainy Season 1970					Dry Season 1970			
Ammonia NH_3	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Iron Fe	2.0	1.8	5.3	3.2	7.1	4.0	3.3	3.0	1.5	1.6		
Manganese Mn	neg.	neg.	0.4	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Residue	140	188	139	281	417	301	361	69	309	96		
Chloride Cl	21	22	15	13	16	22	43	14	11	27		
Nitrite NO_2	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Nitrate NO_3	pos.	pos.	neg.	neg.	neg.	neg.	neg.	neg.	neg.	neg.		
Sulfate SO_4	neg.	neg.	6	TU.	TU.	TU.	TU.	TU.	TU.	7		
pH	7.4	7.4	6.8	5.9	3.3	6.3	8.0	6.3	6.3	7.4		
Color	CL.	CL.	TU.	TU.	TU.	TU.	TU.					
Bic. hardness D.H.	2.5°	3.1°	1.1°	0.9°	0.9°	0.8°	3.0°	0.9°	0.9°	1.5°		
Tot. hardness D.H.	2.6°	3.1°	1.1°	0.9°	1.0°	0.3°	4.0°	0.9°	0.9°	1.6°		
$KM_n\ O_4$ demand	24	26	60	72	83	75	59	28	31	30		
H_2S	neg.						neg.					
				Bacteriological Examination								
Plate counts	300	200	300	600	730							
Most probable/single number	0	0	0	10,000	35,000							
Eichmann/test	pos.	0	0	pos.	pos.							

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CHAPTER 5

WATER TREATMENT

NEED FOR TREATMENT

Standards for water quality for public water supplies were discussed in Chapter 3, Water Quality Requirements, and the quality of available water was discussed in Chapter 4, Sources of Supply. This chapter concerns the need for treatment of existing raw water to provide an end product which is hygienically safe, esthetically attractive, and palatable.

In the selection and development of raw water sources for small water supplies, levels of necessary treatment are of prime importance.

The consideration of levels of treatment to be provided is defined in the World Health Organization Monograph Series No. 42, "Water Supply for Rural Areas and Small Communities." Briefly, these are as follows:

Source and Treatment:

First-priority consideration:

Water which requires no treatment to meet bacteriological, physical, and chemical requirements* and which can be delivered to the consumer by a gravity system should be given first consideration. This would usually be limited to springs and protected drainage areas. Such a system requires no treatment and no pumping, and is therefore ideal from the point of view of maintenance, which is thus reduced to an absolute minimum.

Second-priority consideration:

Water which requires no treatment to meet bacteriological, physical, and chemical requirements but which must be pumped to consumers would be the second choice. Well supplies fall within this category.

Third-priority consideration:**

Water which requires simple treatment before it can meet bacteriological, physical, or chemical requirements but which can be delivered to the consumer through a gravity system should be given

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* These requirements depend on water quality standards applied by each country, and will vary from one place to another.

** Depending on local circumstances, this could be a second priority.

third-priority consideration. Simple treatment is considered to be limited to 1) storage which would provide plain sedimentation and some reduction in bacteria; 2) chlorination without the use of a mechanically operated chlorinator; and 3) slow sand filtration; or a combination of these.

Fourth-priority consideration

Water which requires simple treatment, as mentioned above, and which must be delivered to the consumers by pumping would obviously be the most expensive choice to make.*

Because of the limitations imposed by topography, salinity, population concentrations, and the like, the water supplies in the more populated areas of Surinam generally fall into the fourth priority consideration as defined above. Since this is the most expensive choice, it behooves the designer to exercise imagination and ingenuity in effecting as many economies as possible in the design of treatment and distribution facilities in order to keep the water supplies within the financial reach of as many communities as possible.

TREATMENT OF GROUND WATER

The ground water sources supplying the existing systems in Surinam as well as the potential planned systems are of such physical and chemical quality as to require only a relatively simple method of treatment. Bacteriological pollution of ground water has not constituted a major problem as yet.

Principally, the need for treatment, therefore, is limited to aeration, rapid sand filtration, and "shell" filtration or "polishing."

In order to meet the relatively high standard for potable water in Surinam the ground water treatment is related to iron, carbon dioxide, hydrogen sulfide, manganese, and methane, which are found in sufficient quantities to merit special treatment. Iron is removed by revising the originally acidic condition of the water. Carbon dioxide is mainly responsible for causing the acidic condition, and aeration is applied to adjust this condition. During this degassification of the water other objectionable gases such as H_2S and methane are removed, and pH correction is effected to a degree.

Aeration as a method of iron oxidation and degassification is considered useful when applied to typical Surinam ground water because of the following:

*Depending on local circumstances, this could be a second priority.

1) Simplicity

The mechanics of aeration is simple, although the process itself is multivarious. It does not involve chemicals which have to be imported, nor is special feeding equipment required.

2) Composition

Because Surinam ground water has a relatively constant composition and does not contain inhibiting compounds, iron removal by other means would be difficult and would require treatment with chemicals. The dissolved iron present in the raw water is unstable. Ground water which is clear when drawn will turn cloudy and precipitate when the sample is exposed to air for a period of time.

3) Standardization

This aeration method is already familiar to treatment plant personnel and has been successfully applied in this country for years, justifying the continuance of this type of application for practical purposes.

4) Maintenance

The well-designed aeration system will operate relatively trouble free for long periods without the need of highly-trained, skilled maintenance workers.

5) Corrosion Control

While providing atmospheric oxygen for the oxidation of iron and the liberation of gases, the aeration process serves to prevent corrosion.

"Shell" filtration or "polishing" is used (in addition to aeration) in Surinam as a final step in the ground water treatment process for several reasons. As a treatment medium it promotes stabilization and effects final reduction of the CO_2 as well as contributing to pH correction. Shells are readily available in the country in large amounts and need only washing prior to use. These shells are the residue of countless fresh water mollusks and occur in shell banks in numerous locations in Surinam.

Therefore, when treatment by aeration and sand filtration alone tend to be inadequate, "shell" filter is included as an additional means of producing high quality drinking water. "Shell" filtration as a source of calcium carbonate has the following specific advantages:

- adjusts the pH up to a maximum of 7.8
- limits the CO_2 contents to approximately 2.0 mg/liter

TREATMENT OF SURFACE WATER

River Supplies

Treatment of surface water in Surinam can be traced to 1926 in which the Surinaamsche Bauxite Maatschappij began treating Cottica River water at Moengo through chlorination and filtration. The need for heavy chlorination in order to produce an acceptable drinking water let to the discontinuance of the treatment plant operation. A new plant was constructed in 1937 but this, too, was discontinued after adequate ground water supplies were developed. However, with the average daily demand reaching approximately 100,000 gallons/day by 1954, the need existed again for treatment of river water. Subsequently a treatment plant was built which incorporated the following steps:

1. Screening at intake
2. Addition of chemical flocculent
3. Mixing
4. Flocculation
5. Sedimentation
6. Rapid sand filtration
7. pH adjustment
8. Chlorination

This plant is still in operation.

In 1958 the water treatment plant for the Ministry of Health Leprosarium at Groot Chatillon was placed in operation by the Public Health Division of SABTS. Serious problems in water transport to the leprosarium by barge had been encountered during dry seasons, and these problems were alleviated by construction of the new plant. The Ministry of Health still operates this small (30 to 40 m^3/day) plant utilizing alum, prechlorination, and rapid sand filtration for treatment of Suriname River water. Pressure is developed through use of a hydrophor.

On only one occasion in the history of operation of the Groot Chatillon plant was the chloride level in the Suriname River of a sufficiently high level and for a sufficiently long period to be troublesome.

In both of the above treatment plants the relatively high color and organic level in the raw water does not seem to pose a problem.

A small rapid sand filtration plant is operated by one of the aluminum companies and utilizes Suriname River water with an intake at a point upstream from Paramaribo (approximately 88 km).

The possibility of utilizing slow-sand filtration for treating surface water was given consideration early in the life of the project. Because slow-sand filters are effective in removing organic substances and are efficient in retaining bacteria and suspended matter, it was felt that the surface water in Surinam would lend itself to this type of treatment.

The decision was based on the need to develop a system of water treatment which would satisfy the following conditions:

1. Economy of construction and operation
2. Requiring little skilled labor for design, construction, operation, and maintenance
3. Strong, simple construction
4. Acceptability of final product

The advantages and disadvantages of slow-sand filters are listed in "Operation and Control of Water Treatment Process," by Charles R. Cox, WHO Monograph Series No. 49, as follows:

SLOW SAND FILTERS

The advantages of slow sand filters may be summarized as follows:

1. There is no need for coagulation facilities.
2. Equipment is simple and does not need to be imported.
3. Suitable sand is readily secured.
4. Supervision is simple.
5. The effluent is less corrosive and more uniform in quality than chemically treated waters.
6. They give effective bacterial removal.

The disadvantages as compared to rapid sand filters are as follows:

1. A large area is required, with correspondingly large structure and volume of sand and higher structural costs.
2. They have less flexibility in operation.
3. They are not economical with raw waters having turbidities over about 30 units for prolonged periods, unless preliminary plain sedimentation will secure such turbidities in the settled water.
4. They are less effective in removing color.
5. They give poor results with water of high algal content, unless pretreatment is practiced.

In view of the above disadvantages, it was decided to construct small, portable test filters which could be installed in several locations in Surinam in which the surface water exhibited different characteristics. Figure IV-4 shows the main features of the test filters. The vertical height of the test filter was determined in such a manner as to approximate the height of an actual slow sand filter. Shortcircuiting was countered by painting the inner surfaces of the test filter with INERTOL, and dusting the wet paint with sand in order to roughen the surface.

Tests were conducted at the following locations:

Brownsweg (van Blommenstein Meer)
Uitkijk (Saramacca Canal)
Saramacca Doorsteek (Saramacca Canal)
Groot Chatillon (Suriname River)
Blaka Watra (Suriname River)

In all cases in which test runs were initiated, the length of run was continued for a maximum of 10 days in order to allow building-up of the "Schmutzdecke" or biological layer.

A number of variables were introduced into the test program, including selecting filter sand from several different locations. This allowed variation of grain sizes and uniformity coefficients. Graphs showing typical sieve analyses are included as Figures IV-5 and IV-6. For one series of tests, a mixed-media in which fractured shells were present was utilized. For another series, activated charcoal was used in the filter media.

Because sufficient time and manpower were not available for an exhaustive and definitive test program, the program could not be extended over an unlimited period of time at each test of location, nor could an exact evaluation of all variables be made.

However, the following positive basic results were obtained by varying the media:

1. Definite reduction in color level
2. Effective filtration of turbid waters
3. Satisfactory pH adjustment
4. Reduction in iron level
5. Reduction of bacteriological counts

It is of interest to note that, although it was not deemed necessary in the process, except as a precautionary measure if slow sand filtration is used in public water supplies, the subsequent chlorination of some of the samples of filtered water in which minimal color was still evident resulted in "bleaching" of the color after a 15-minute retention period.

Further testing was conducted with a different type of installation in order to determine the effect of algal growth resulting from the operation of filters exposed to sunlight vs. covered filters.

Two filters were constructed, both of 15 cm Ø PVC pipe and both containing 1.35 m of Coesewijne sand (see Figure IV-7). This sand has an effective diameter, d_{10}, 10% passing, of 0.2 mm and a 90% passing diameter, d_{90}, of 1.4 mm.

The average chemical analysis of the raw river water is shown on Table IV-11.

Although the "uncovered" filter (transparent cover) tended to produce shorter filter-runs due to filterbed clogging as a result of algae growth, a beneficial influence on effluent quality was indicated. This was attributed to increased oxygen production by the algae.

The raw water sampling site which was selected for this testing was deliberately chosen for its negative aspects, particularly the variations in organic matter loading and sediment loading. Although an intake would not be installed at the sampling site, the test program indicated that the river water could be treated through slow sand filtration, utilizing pretreatment and postchlorination.

Reviewing the disadvantages of slow sand filters which were listed earlier, the large areas of land required are readily available in Surinam; flexibility of operation would not be required since the raw water varies little in quality over long periods of time. When the intake is sufficiently far upstream, the turbidity and color of the raw water does not seem to pose a problem, and the algal content of raw water effects has no seeming detrimental effect.

In view of the encouraging results as demonstrated above, a slow-sand filtration plant was installed as part of the Brownsweg water supply.

The incorporation of slow-sand filtration into major water supply systems in Surinam will depend upon the continuing availability of ground water. Since it appears that supplying ground water to communities such as Kwatta-Leidingen and Pad van Wanica West will probably eventually constitute a "mining" operation, the need for utilizing surface water sources will occur in the distant future, but not within the master planning projection period.

FLUORIDATION

Following a PAHO seminar in Puerto Rico in 1968, the Government of Surinam created a committee on fluoridation. The committee, composed of the Chief, Water Supplies Section, Ministry of Public Works; the Manager,

TABLE IV-11

RAW WATER AT UITKIJK (SARAMACCA RIVER)

	Cl mg/1	umho $.10^{-6}$	pH	Total Solids mg/1	Color mgr Pt/1	$KMnO_4$ used	NO_3 mg/1	NH_4 mg/1	P_2O_5 mg/1	O_2 mg/1	Temp. 0^oC.
Average between 15-11-1971 and 18-2-1972	15	82	6.9	14	95	31	0.5	0.54	13.9	6	28

Surinam Water Company; and the Chief Dentist of the Ministry of Health, was to advise the Government concerning the feasibility of fluoridation in the water supplies of Surinam. Circumstances seem to have developed which have impeded this function of the committee.

Although the study of fluoridation was not a defined task in this project, close cooperation was provided by project staff during the visits of two PAHO consultants in fluoridation. Attention is drawn to the "Report on Fluoridation of Drinking Water in Urban and Rural Areas of Surinam," which was prepared by the consultants.

Fluoridation equipment was not included in the designs prepared in this project, since the Government is interested in first gaining experience with fluoridation in the smaller water supplies, which are to be built with UNICEF assistance.

CHAPTER 6

INTERIM WATER SUPPLIES

STATUS OF EXISTING WATER SUPPLIES

As of beginning 1972 the following status existed with regard to community water supply systems in Surinam:

Existing Operational Systems

1. Paramaribo, Albina, and Nieuw Nickerie (Surinam Water Company)
2. Koewarasan (Ministry of Rural Government and Decentralization)
3. Meerzorg (Surinam Water Company)
4. Groningen-Tambaredjo (Surinam Water Company)
5. Brokopondo (Surinam Water Company)
6. Klaaskreek (Surinam Water Company)
7. Brownsweg (Surinam Water Company)
8. Alliance (Ministry of Agriculture)
9. Totness (Ministry of Agriculture)
10. Kampong Baroe (Ministry of Agriculture)
11. Groot Chatillon (Ministry of Health)
12. Paranam (Suralco)
13. Moengo (Suralco)
14. Afobaka (Suralco)
15. Onverdacht (Billiton Mij.)
16. Smalkalden (Billiton Mij.)
17. Mariënburg (Cultuur and Rubber Mij.)
18. Wageningen (SML)
19. Wonoredjo (Ministry

Systems under Expansion

1. Paramaribo (Surinam Water Company)
2. Groningen
3. Brokopondo
4. Koewarasan

Systems under Planning

1. Brokopondo (expansion)
2. Coronie (elevated storage)
3. Meerzorg and Peperpot (expansion)
4. Kwatta-Leidengen
5. Kwakoegron
6. Groot en Klein Henar
7. Corantijn and van Drimmelen Polder
8. Paradise
9. Calcutta-Tijgerkreek West
10. Nieuw Lombe-Mooie Kreek
11. Pad van Wanica West
12. Uitkijk-Jarikaba
13. Jagtlust-Nw. Amsterdam-Voorberg, Alkmaar Zoelen, Mariënburg
14. Spieringshoek-Wederzorg and Katwyk
15. Tamanredjo
16. Leliendal

It was anticipated that, in addition to the above, the Supply Group I Systems would commence to be constructed as the final designs and financing would become available.

PRESENT STATUS OF SUPPLY GROUP I SYSTEMS

The original listing of communities in the Request and Plan of Operation for the UNDP(SF) Project delineated population groups according to existing built-up areas. Based on field experience, a regrouping of supply systems for these communities has been effected according to the following:

Supply System	Interim (1987) Approximate Program Design Population
Kwatta-Leidingen	25,000
Pad van Wanica West	25,000
Commewijne	50,000

The design populations shown above indicate that, for Supply Group I areas, the systems will be relatively small; but, more important, the systems can be designed on the basis of populations in multiples of 25,000 persons. This, in turn, provides the opportunity to design the community water supply systems on the basis of basic standard design elements.

Further, standardization of treatment methods and operations provides flexibility and promotes efficiency, since operating personnel can be transferred easily from one plant to another, and interchangeability of equipment allows reduced inventories of spares.

In major urban water supplies, of course, this approach is not easily implemented since each element of a large system presents unique and individual problems. The standardized approach also emphasizes the serious responsibility facing the designer, who cannot sacrifice good engineering judgement for the sake of standardization. Because the Interim Program will consist essentially of a number of similar systems, it follows that many design features will be repeated several times. In the selection of these design features, therefore, the designer must carefully select and compare in order to develop a design which accomplishes the planned objective at the least cost.

In the Supply Group I systems, the standardization is basically connected with the treatment plant elements, because the individual distribution systems tend to differ from each other.

Referring to the Supply Group I areas, the original grouping in Appendix I of the Plan of Operation listed (in addition to Kwatta-Leidingen) Mariënburg, Tamanredjo, Nieuw Amsterdam and Voorburg, Spieringshoek, Jagtlust, Alkmaar, and Meerzorg. These latter have all been incorporated into the Commewijne Supply Group. Houttuin is not being considered, since the area is very scarcely populated and shows little reason for further accelerated expansion during the master planning period. Uitkijk and Jarikaba are considered with the Kwatta-Leidingen Project. Koewarasan is an existing supply undergoing augmentation at present which will provide for the next 15 years of growth. Paranam and Onverdacht are "company towns" of the aluminum industries in Surinam, and, although the project staff has worked closely with the water utility and geology staff in the companies, it was made known that the water supply needs of these communities will be met by the aluminum companies themselves.

In the case of Pad van Wanica West, this represents a new population group which was somehow not included in the original request or Plan of Operation and is a relatively large geographical area of medium population concentration.

PRESENT STATUS OF SUPPLY GROUP II SYSTEMS

The communities listed under the Supply Group II designation consist of small semiurban or rural settlements, generally located in the coastal region. The supply systems for these communities are as follows:

Supply System	Interim (1987) Program Design Population
Groot en Klein Henar	6,300
La Vigilantia and Smalkalden	14,660
Paradise	15,190
Corantijn and van Drimmelen Polder	15,190
Calcutta and Tijgerkreek	7,050
Domburg	12,500

It is noted that, except in two cases, the design populations of the above systems can be regarded as multiples of approximately 7,000 persons. This, again, provides the designer with the opportunity to apply a standardized approach for the production elements of the systems.

None of the above communities has a piped water supply at present. However, these communities are included in a project which makes use of UNICEF assistance in construction. The UNDP(SF) Project provided the preliminary engineering and feasibility studies required for securing the UNICEF assistance.

PRESENT STATUS OF SUPPLY GROUP III SYSTEMS

While there are a number of hand-dug wells in use, this group of small communities depends almost entirely upon the use of river water for domestic needs. None of the Supply Group III communities have either treatment plants or distribution systems. Because surface water sources are utilized, the systems for these villages are included in the model or standard design which follow.

STANDARD DESIGN FOR SLOW SAND FILTRATION (see Figures IV-8, -9, and -10)

Applicability

1. As sole treatment of clear water from lakes.

2. When preceded by plain sedimentation as treatment of water with little turbidity from creeks and upland rivers.

3. When preceded by chemical coagulation and followed by chlorination as treatment of more turbid waters from lowland rivers.

Unfit when the raw water has a high turbidity, or when difficult to remove an excessively large amount of color of vegetable origin.

Design Criteria

Design rates:

Maximum daily to average daily consumption	1.4
Maximum hourly to average hourly	1.8
Maximum capacity to average daily	2.5
Plant operating time	12 hours
Storage as percentage of average daily consumption:	
Ground level	50
Elevated level	5.5 (or sufficient to allow constant filtration rate during operation period)
Maximum allowable filtration rate	0.25 m/hour
Minimum size filtering unit	10 m^2
Minimum pipe size	2"

Raw water supply by perforated pipe, outflow resistance maximum 0.45 m, $\pm$ 1 m above maximum water level settled water discharge via opening, 0.8 m width, discharge 3.8 $m^3/m^2/hr$.

Filter

Drainage system to be perforated bricks with layers of graded gravel, ground discharge via telescopic tube, lowered every two to three days to keep raw water level between 0.5 and 0.1 m below top of filterbox.

Clear Water Storage

With guide wall to prevent short circuiting after chlorination (0.5 - 1.0 ppm).

Minimum volume 1.5 x 6 x 1.8 = 16 m^3, minimum detention time five hours.

Design Characteristics for 500 and 2,000 Persons

	Design Production	
	500	2,000
Average daily demand at 50 l/c/d	25 m^3/day	100 m^3/day
Demand in maximum (140%) day	35 m^3/day	140 m^3/day
Pumping rate for 12 hr. day	3 m^3/hr	12 m^3/hr
Raw water pumps*	2x3 m^3/hr	2x12 m^3/hr
Tray settling tanks*		
Surface area	9 m^2	15 m^2
Surface loading	0.2 m/hr	0.2 m/hr
Detention time	8 hrs	8 hrs
Filters*(areas)	2x12 m^2	2x32 m^2
Filtered water demand for maximum (250%) hour	5.3 m^3/hr	21 m^3/hr
Filtered water pumps*	2x5 m^3/hr	2x14 m^3/hr
Chlorination (minimum)	0.5-1.0 ppm	0.5-1.0 ppm
Clear water storage		
Low level (minimum)	12.5 m^3	50 m^3
Elevated (minimum)	1.4 m^3	5 m^3

* One standby unit

CHAPTER 7

RECOMMENDED WATER SUPPLIES PROGRAM

MASTER PLAN 1972-2000

The development of water supply systems for all Supply Group areas through the Interim Planning Period (1972-1987) has been described in general terms in an earlier section. In evolving the master plan the investigation of the needs for water supply systems was conducted in the context of meeting those needs through preliminary identification of optimum plans and organizations.

Scope

The master plan for the provision of water supplies covers the specific time period from 1972 through 2000 and, in the case of Supply Group I systems, covers specific areas.

In developing the master plan, constraining or governing factors which affected long-term programming had to be kept in view. For example, the proposed formation of the water authority did not occur, and the responsibility for municipal supplies continued to be vested in the Surinam Water Company, while the administration of rural water supplies remained with the Ministry of Rural Government and Decentralization.

The financial requirements for the master plan program required a special approach because of the existing governmental structure of the Kingdom of the Netherlands. Thus, the sources of financial assistance were the Netherlands Five-Year Plan, European Economic Community, UNICEF grants, and the Surinam Government budget.

In addition to the governmental and financial aspects of master planning, the relationships between the provision of water supplies and other high priority essentials had to be taken into account since these amenities all affect the social and cultural life of the people.

PRIORITY LISTING OF WATER SUPPLIES

The master plan for the construction of water and sewerage systems in Surinam has been prepared in accordance with an estimate of the relative urgency of need for the proposed systems, taking into account the earlier preliminary planning which has been executed in the UNDP/SF project consistent with the Government's definition of immediate priorities.

The relative priorities established in the Group I water systems are as follows:

1. Kwatta-Leidingen with Uitkijk and Jarikaba
2. La Vigilantia and Smalkalden
3. Pad van Wanica West
4. Commewijne area, consisting of Meerzorg, Jagtlust, Lust and Rust, Belwaarde, Voorburg, Nieuw Amsterdam, Zoelen, Mariënburg, Leliendal, Spieringshoek, Wederzorg, Katwijk, Támanredjo, and Alkmaar
5. Domburg

The relative priorities established for the Group II water systems are as follows:

1. Nieuw Lombe and Moeje Kreek
2. Groot en Klein Henar
3. Paradise
4. Corantijn and van Drimmelen Polders
5. Calcutta and Tijgerkreek

The water supply systems of Paranam, Moengo, and Onverdacht will be expanded as required by the aluminum industries responsible.

Because of the problem of intermittent supply, the plans for the system of Brownsweg were designed changing the source from Macami Creek to van Blommensteinmeer to reduce the length of the supply main from 7 to 2 km and provide continuous service. The plant design consisted of slow sand filtration and chlorination with gravity flow to existing storage. The treatment plant and supply main construction have been completed. The cost of the completed project was Sf.50,000.

Designs were also completed for the Group II system for Nieuw Lombé and Moeje Kreek and construction of this project is also under way. The cost estimate for this project is Sf.40,000.

The Group I system for La Vigilantia was also designed and a production well located and drilled about 1 km south of the community to be served. The cost estimate for this project due to the distance of the supply main is Sf.300,000.

In all priority listings it is fully realized that changing conditions, new developments, the location of industrial sites, or housing construction can radically change the relative need for water supply. Therefore, all priority listings should be reviewed annually to determine if the previously planned program is still valid.

DESCRIPTION OF FACILITIES

Group I

The Group I projects as designed under the UNDP/SF program consist of the following elements:

Kwatta-Leidingen System

The final design (see Plan IV-1) for this area consists of a well supply, aeration, sand filtration ground, clear water storage, and elevated storage. The designed capacity is for 2,250 m^3/day for 1987 with expansion to 4,450 m^3/day for the year 2000. This plant is now under construction.

The distribution system plan calls for the installation of 74 km of line for 1987 and an additional 24.6 km by the year 2000. The plan provides for the eventual supply to Uitkijk and Jarikaba. The Uitkijk supply will require 6.4 km of main by 1987 with the replacement of 3 km of 4" with 6" main by the year 2000.

Details of the estimate for construction costs are as follows:

Kwatta-Leidingen Treatment Plant and Distribution System

Well Supply		/1987/
1. Treatment plant, including reservoirs, pump station (no standby power at site)	Sf.	400,000
2. Distribution system, including house connections		800,000
Subtotal	Sf.	1,200,000
3. Pump station, reservoir, Uitkijk-Jarikaba (no standby power at site)		62,000
4. Distribution system, including house connections (only Uitkijk)		90,000
5. Main to serve Jarikaba project from pump station (distribution system will be installed by others)		40,000
Total for year 1987	Sf.	1,392,000
1. Expansion of existing facilities treatment plant, including diesel generator for standby power	Sf.	345,000
2. Distribution system, including house connections		590,000
3. Expansion of existing pump station Uitkijk, Jarikaba		20,000
4. Distribution system, including house connections (Uitkijk only)		40,000
Total added for year 2000	Sf.	995,000

N.B. All construction costs based on 1972 unit cost prices.

Pad van Wanica System

The plans for the Pad van Wanica system (see Plan IV-2) are much the same as previously noted for the Kwatta-Leidingen project with treatment plant and well located on Helena Christina Weg. The plant is designed to produce 2,250 m^3/day for the period to 1987, with expansion to 4,450 m^3/day by the year 2000.

The distribution system plans call for the installation of 130 km of main for the period to 1987 with an additional 31.5 km for the year 2000.

Details of the estimate for construction costs are as follows:

Pad van Wanica Treatment Plant and Distribution System

Well Supply		/1987/
1. Treatment plant, including reservoirs, pump station, with portable diesel generator to also serve K-L project	Sf.	480,000
2. Distribution system, including house connections		1,720,000
Total for year 1987	Sf.	2,200,000
1. Expansion of existing facilities treatment plant	Sf.	345,000
2. Distribution system, including house connections		540,000
Total for year 2000	Sf.	885,000

N.B. All costs are based on 1972 unit cost prices.

Commewijne System

The population of this area is scattered in small communities over a wide territory mainly along the east bank of the Suriname River from Meerzorg to Nieuw Amsterdam and along the south bank of the Commewijne River upstream from Spieringshoek. A scattered population along the East-West Highway has its greatest concentration at Tamanredjo.

Presently a small distribution system supplied by a well provides water to a small part of the Meerzorg area. The demand for water in the Meerzorg-Peperpot area is estimated to be 1,500 m^3/day by 1987. For the entire Commewijne area the requirements for 1987 are 5,000 m^3/day.

With the exception of the limited supply at Meerzorg no other satisfactory ground water was found in the Commewijne area. As a result three studies of alternate sources were made.

First a ground water source was considered east of the Commewijne supply area. The ground water along the East-West Highway at Orleana Kreek was found to be unsatisfactory. The Zanderij aquifer southward was at least 15 km south of the East-West Highway across intervening swampy terrain with no communication. The ground water at Morico Creek west of the Commewijne River was found to be suitable; thus, the first design (see Plan IV-3) was based on that source and the costs are estimated as follows:

	1987	2000
Treatment plant, including standby power	Sf. 950,000	500,000
Supply line to Commewijne area	278,000	893,000
Booster station for supply line	162,000	90,000
Distribution system, including house connections	2,655,000	1,120,000
Meerzorg booster station and reservoirs, including standby power	160,000	106,000
Marienburg booster station and reservoirs, including standby power	182,000	50,000
Belwaarde booster station, including standby power	-	235,000
Alkmaar booster station, including standby power	-	237,000
	Sf.1,387,000	3,231,000

The second study (see Plans IV-4 to -9) was based on a ground water supply found to be available at Rijsdijkweg (see Volume III for details). This plan required a treatment plant and long supply main, including a crossing of the Suriname River. The cost estimates for this proposal are as follows:

	1987	2000
Well supply and treatment plant	Sf. 1,667,000	970,000
Diesel generators (if no other power available)	100,000	50,000

Main supply line	4,215,000	-
River crossing	185,000	-
Booster station at Leiding 9A	-	150,000
Distribution system, Commewijne area, including house connections	2,165,000	275,000
	Sf. 8,332,000	1,445,000

The third study considered the use of surface water from the Saramacca River at Santigron. This approach (see Plans IV-10 to -14) also required a treatment plant, long supply lines, and a crossing of the Suriname River. The estimates for this proposal are as follows:

	1987	2000
Treatment plant	Sf. 1,000,000	515,000
If no regular power supply add generator (also standby)	100,000	50,000
Main supply line	3,315,000	
River crossing	185,000	
Booster station at Leiding 9A		150,000
Distribution system, Commewijne area, including house connections	2,165,000	275,000
	Sf. 6,765,000	990,000

With the relatively high costs involved in all three of the studies, consideration was given to the use of the somewhat limited supply at Meerzorg.

The hydrogeological studies (see Volume III) indicated the availability of 10 million m^3 of fresh water from this source which at 1987 consumption rates would be sufficient for Meerzorg for 20 years (average 0.56 million m^3/year) or to supply Meerzorg, Lust and Rust, Belwaarde, Nieuw Amsterdam, Voorburg, Mariënburg, Tamanredjo, Leliendal, and Zoelen for 10 years (average 1.4 million m^3/year).

The cost estimates for the use of this source for Meerzorg are as follows:

		1987	2000
1. Treatment plant, including pumps, ground and elevated storage	Sf.	345,000	-
1972 expansion		90,000	-
Distribution system		94,700	26,175
House connections		26,175	175,000
	Sf.	555,875	201,175

In order to expand the system to supply the area north of Meerzorg to Nieuw Amsterdam and Zoelen the following estimated additional costs are required:

		1987	2000
2. Meerzorg-Zoelen transmission main	Sf.	586,300	-
	Sf.	586,300	
a) Lust and Rust distribution system		17,250	-
House connections		4,300	6,000
	Sf.	21,550	6,000
b) Belwaarde distribution system		-	18,000
House connections		8,000	-
	Sf.	8,000	18,000
c) Voorburg distribution system		-	-
House connections		15,000	23,000
	Sf.	15,000	23,000
d) Nieuw Amsterdam distribution system		55,950	2,185
House connections		14,000	85,000
	Sf.	69,950	87,185
e) Zoelen distribution system		-	-
House connections		28,000	104,000
	Sf.	28,000	104,000

		1987	2000
3. Zoelen-Mariënburg transmission main	Sf.	59,500	-
	Sf.	59,500	-
a) Mariënburg distribution system		20,075	15,345
Mariënburg house connections		6,450	143,000
	Sf.	26,525	158,345
4. Mariënburg-Leliendal transmission main	Sf.	123,750	-
	Sf.	123,750	-
a) Leliendal distribution system		5,900	
House connections		1,480	10,000
	Sf.	7,380	10,000
5. Leliendal-Spieringshoek transmission main	Sf.	85,000	-
	Sf.	85,000	-
a) Katwijk distribution system		14,000	-
House connections		20,000	5,000
	Sf.	34,000	5,000
b) Wederzorg distribution system		14,325	-
House connections		6,000	5,000
	Sf.	20,325	5,000
c) Spieringshoek distribution system		-	-
House connections		5,000	5,000
	Sf.	5,000	5,000

Expansion of the system eastward along the East-West Highway would require the following incremental expenditures:

		1987	2000
6. Meerzorg-Tamanredjo transmission main	Sf.	184,800	-
	Sf.	184,800	-

		1987	2000
a) Tamanredjo distribution system	Sf.	22,300	28,850
House connections		6,076	64,000
	Sf.	28,376	92,850

The final construction required to complete the master plan provides for transmission mains and house connections with the following estimated costs:

		1987	2000
7. Tamanredjo-Alkmaar transmission main	Sf.	120,000	-
a) Alkmaar distribution system		100,000	-
House connections		30,000	-
	Sf.	250,000	-
8. Belwaarde-Alkmaar transmission main	Sf.	160,000	-

At that point where the total water withdrawn from the Meerzorg wells results in production of a water of unacceptable quality it will be necessary to make use of one of the more distant supplies. As a first step it would be possible to expand the Kwatta-Leidingen plant and provide for transmission mains to the Commewijne area with the following estimated costs:

		1987	2000
Expansion of Kwatta-Leidingen treatment plant	Sf.	950,000	-
Transmission main to river		1,705,000	-
River crossing		185,000	-
Booster station at river crossing		-	330,000
Convert Meerzorg plant to booster station		30,000	65,000
	Sf.	2,870,000	395,000

When the expansion at the Kwatta-Leidingen plant becomes insufficient to meet the needs of the Commewijne area, implementation of a new supply will be required with the following estimated costs:

Alternate I

Convert part of Kwatta-Leidengen plant to booster station	Sf.	50,000
Santigron transmission main to Kwatta booster station		1,610,000
Santigron surface water treatment plant		1,810,000
Total	Sf.	3,470,000

Alternate II

Convert part of Kwatta-Leidengen plant to booster station	Sf.	50,000
Rijsdijkweg transmission main to Kwatta booster station		2,510,000
Treatment plant for ground water at Rijsdijkweg		1,767,000
Total	Sf.	4,327,000

A summary of the estimated costs for the Commewijne system is listed as follows:

		1987	2000
1. Meerzorg	Sf.	555,875	201,175
2. Meerzorg Zoelen		586,300	-
a. Lust and Rust		21,550	6,000
b. Belwaarde		8,000	18,000
c. Voorburg		15,000	23,000
d. Nieuw Amsterdam		69,950	87,185
e. Zoelen		28,000	104,000
Subtotal	Sf.	1,284,675	439,360
3. Zoelen-Mariënburg transmission main	Sf.	59,500	-
a. Mariënburg distribution system, including house connections		26,525	158,345
Subtotal	Sf.	1,370,700	597,705

		1987	2000
4. Mariënburg-Leliendal transmission main	Sf.	123,750	-
a. Leliendal distribution system, including house connections		7,380	10,000
Subtotal	Sf.	1,501,830	607,705
5. Leliendal-Spieringshoek transmission main	Sf.	85,000	-
a. Katwijk distribution system and connections		34,000	5,000
b. Wederzorg distribution system and connections		20,325	5,000
c. Spieringshoek distribution system and connections		5,000	5,000
Subtotal	Sf.	1,646,155	622,705
6. Meerzorg-Tamanredjo transmission main	Sf.	184,800	-
a. Tamanredjo distribution system and house connections		28,376	92,850
Subtotal	Sf.	1,859,331	715,555
7. Alkmaar-Tamanredjo transmission main	Sf.	120,000	-
a. Alkmaar distribution system and house connections		130,000	-
Subtotal	Sf.	2,109,331	715,555
8. Belwaarde-Alkmaar transmission main	Sf.	160,000	-
Subtotal	Sf.	2,269,331	715,555
Expanded Kwatta-Leidingen supply	Sf.	2,870,000	395,000
Subtotal	Sf.	5,139,331	1,110,555

The final move toward the basic and long-term supply, with consideration of the operating costs noted in the next paragraph, will require construction of one of the two alternatives previously noted.

		2000
Alternate 1 - Surface water Santigron	Sf.	3,470,000
Alternate 2 - Ground water from Rijsdijkweg		4,327,000

Detailed studies for the unit costs of producing water from both surface and ground water sources were made. The results of this work indicated that the relative costs were as follows:

		Cost per m^3	
		To 1987	To 2000
Ground water	Sf.	0.07	0.085
Surface water		0.15	0.165

The comparison is also noted in Figures IV-11, -12, and -13.

The conclusions reached as a result of the studies noted above are that the Meerzorg supply should be used as long as it is available; that reservoirs, distribution and supply mains be installed as shown on the master plan; and that when required the more distant supplies be developed in the most economical manner available at that time, consistent with general development of the area.

Present calculations indicate that the additional capital costs of ground water treatment would be amortized in approximately 10 years by the savings in operating costs noted for ground water as compared with operating costs for surface water production.

The use of treatment plants to produce fresh water from brackish water has not been considered in this report because of the high cost involved with the present state of technical development.

However, there are many countries with the same problems of brackish underground supplies as are found in Surinam which are conducting substantial research programs to improve the treatment processes and reduce operating costs. It is possible that by the time that the Meerzorg supply is exhausted, treatment of the brackish water may provide an economic alternative to the long transmission mains and river crossing noted in the Master Plan. Future technical developments in the field of membrane filter treatment or any other method of treating brackish water should be closely followed.

Group II

The Group II systems with partial financing awarded by UNICEF have been designed (see Plan IV-15) and estimated as follows:

Groot en Klein Henar is designed (see Plan IV-15) for a ground water source, with the treatment plant consisting of aeration, sand and shell filtration, and ground and elevated storage with about 40 km in the distribution system by 1987 and an additional 32 km by year 2000.

The costs estimated for this project are as follows:

		1987	2000
1. Treatment plant	Sf.	143,000	50,000
2. Distribution system, including house connections		285,000	180,000
	Sf.	428,000	230,000

The Paradise system (see Plan IV-16) is designed for a ground water source. Treatment is provided through aeration, sand filtration, and shell filtration. Ground level and elevated storage are also planned; 37 km of distribution piping by 1987 and an additional 17 km by the year 2000 is to be installed.

The estimated costs for this project are as follows:

		1987	2000
1. Treatment plant	Sf.	235,000	70,000
2. Distribution system, including house connections		280,000	65,000
	Sf.	515,000	135,000

Corantijn and van Drimmelen Polders have a system design (see Plan IV-17) providing for a ground water supply, aeration, sand and shell filtration, and ground and elevated storage, with 38 km of distribution mains by 1987 and an additional 15 km by the year 2000.

The estimated costs for this project are as follows:

		1987	2000
1. Treatment plant	Sf.	290,000	100,000
2. Distribution system, including house connections		565,000	240,000
	Sf.	855,000	340,000

The Calcutta and Tijgerkreek system is also designed (see Plan IV-18) for ground water with aeration, sand and shell filtration, and ground and elevated storage, with 13 km of distribution piping required by 1987 and an additional 12 km by year 2000.

The estimated costs for this project are as follows:

		1987	2000
1. Treatment plant	Sf.	145,000	50,000
2. Distribution system, including house connections		197,000	65,000
	Sf.	342,000	115,000

PHASED DEVELOPMENT

As the Master Plan for water supply has a total cost which, from the standpoint of national priorities and available capital funds, must of necessity be spread out over a long period of time, the following phased development is proposed. The sums indicated are based on a continuation of the Netherlands Five-Year Plan support at the same level as in the present five-year plan, with sufficient help from UNICEF and the Government to provide a total of Sf. 5 million for each five-year period from 1976 to 1987. Since the installation of piped water will make the collection and treatment of sanitary wastes even more pressing from the health standpoint, additional funds have been indicated as needed for the development of sanitary sewers or clustered septic tanks for waste disposal at approximately the same level as for water supply construction.

Construction Program

Phase	Period	Water Supplies	Sewerage
Interim	1972-1987	Sf. 16.8 million	Sf. 11.7 million
Ultimate	1987-2000	14.0 million	12.8 million
	Totals	Sf. 30.8 million	Sf. 24.5 million

Interim Phase 1972-1987

This phase contains works for which detailed construction plans have been prepared, or will be in an advanced planning stage, as a result of the UNDP(SF) project. The projects with estimated costs are as follows:

Group I

1972-3	Kwatta-Leidingen, including Uitkijk and Jarikaba	Sf. 1.400 million
1974	La Vigilantia-Smalkalden	.300 million
1974	Pad van Wanica West	2.300 million
1977	Domburg	.500 million
1975-1987	Commewijne system, including Kwatta supply	5.000 million

Group II

1972-3	New Lombé and Mooie Kreek	.040 million
1973	Groot en Klein Henar	.430 million
1974	Paradise	.520 million
1975-76	Corantijn and van Drimmelen Polder	.860 million
1976-77	Calcutta and Tijgerkreek	.350 million
Additional systems with UNICEF assistance		2.600 million
Estimate for expansions of existing systems 15 years x Sf. 0.16 million (approximately) per year		2.500 million
		Sf. 16.800 million

Project's new part of the Master Plan with estimated expansion costs through the year 2000 (ultimate phase).

Kwatta-Leidingen	Sf. 1.000 million
Pad van Wanica	.900 million
Commewijne (Alt. II)	5.500 million
Groot en Klein Henar	.300 million
Paradise	.200 million
Corantijn and van Drimmelen Polder	.400 million
Calcutta and Tijgerkreek	.200 million
Master Plan total	Sf. 8.500 million

Master Plan construction	8.500 million
Construction Group III system	3.000 million
Existing plant expansion	2.500 million
	Sf. 14.000 million

CAPITAL FINANCING

As has been previously noted, the sources of financial assistance are the Netherlands Five-Year Plan, European Economic Community, UNICEF grants, and the Surinam Government budget.

To cover interim phase projects, the sum of Sf.1.2 million has been made available from the first Five-Year Plan 1966-1971 for the Kwatta-Leidingen Project.

From the second Five-Year Plan (1972-1976) the sums of Sf.4.5 million for water supply and Sf.1.7 million for sewerage installations have been made available.

The UNICEF grants have allocated a total of Sf.420,000 ($238,000) for the purchase of materials for Group II projects, and the Surinam Government has allotted Sf.700,000 for labor and materials not covered by the UNICEF grants.

Thus, the total funds now allotted for interim phase construction are as follows:

1966-1971 Five-Year Plan	Sf. 1.20 million
1972-1976 Five-Year Plan	4.50 million
UNICEF	.42 million
Surinam	.70 million
Total	Sf. 6.82 million

On the basis of funds allotted for the first five years of the interim phase, the following construction is proposed:

Kwatta-Leidingen	Sf. 1.20 million
Pad van Wanica West	2.20 million
Uitkijk	.20 million
Commewijne (partial)	1.40 million
Groot en Klein Henar (partial)	.21 million
Paradise	.25 million
Corantijn and van Drimmelen Polder	.46 million
Calcutta-Tijgerkreek	.20 million
La Vigilantia	.30 million
Completion of Corantijn and van Drimmelen Polder system	.40 million
	Sf. 6.82 million

CHAPTER 8

SEWERAGE - GENERAL CONSIDERATIONS

INTRODUCTION

In Part III of this report, the results of detailed investigations are presented in problems involving the provision of adequate potable water supplies. However, to maintain adequate standards of public health, emphasis should be placed not only upon providing readily available suitable water supplies but also upon the removal of waste waters, largely in the form of domestic sewage and industrial liquid wastes, from populated areas and upon ultimate disposal by sanitary means. The removal of storm waters from populated areas is also an important requirement, especially under conditions prevailing throughout the Surinam coastal areas, where drainage by natural means is difficult because of the flat terrain.

In pursuance of the overriding objectives of improving environmental sanitary conditions throughout the project area, preliminary investigations have been made into the existing situation with regard to sewerage. Basic data and design criteria for sewerage have been developed within the frame of reference of the population estimates and water supply proposals previously presented.

EXISTING FACILITIES

Within the project area, with the exception of Paramaribo and the private systems in Moengo and Paranam, there is no sanitary sewerage system in operation. The Paramaribo system, in itself, does not constitute a sanitary sewage collection and treatment facility, in that the waste water collected is the effluent from septic tanks rather than raw sewage. A combined system, carrying this effluent as well as storm drainage, transports the waste water to outfall pumping stations for disposal in the Suriname River (see Figure IV-14). This system has grown with the growth of the city, and because of this it reflects some hydraulic inadequacies.

As Paramaribo continues to expand, the existing combined sewers are sometimes limited in capacity and surcharging results. Further, some of the lines have a definite need for structural rehabilitation.

The use of open canals also creates a problem in that most are hydraulically inefficient and subject to rapid weed growth, and because of this (together with trash dumping in the canals) serious sanitation problems are created.

In connection with the Paramaribo system, the Ministry of Public Works retained a European consulting engineering firm to investigate the sewage and drainage problems in the city and to provide solutions. This effort is still being sponsored through assistance of the European Economic Community. As a result of this study, the consulting engineers observed the following:

a. The central Paramaribo system was functioning adequately.

b. The northern urban area contiguous to the city was in a state of flux, and this seemed to preclude accurate appraisal of the problem, thereby affecting solutions.

c. Collection and treatment in the southern area should be such as to allow only treated effluent to be discharged into the Saramacca Canal.

d. The western area development was such that further economic review was necessary prior to reaching any design conclusions or to making proposals.

e. Whenever possible, separate systems should be installed instead of combined sewers.

Also, in connection with the above study the consulting engineers established the following "urgency" list which refers to Figure IV-14, and is translated as follows:

1. <u>Drambrandersgracht</u>

Here a good working sewerage system could be realized with a small amount of work. A big investment is already made but seems to be very unproductive. Regular flooding problems are presently existing which can be eliminated by the construction of a lift station.

2. <u>Limesgracht</u>

The main line and the discharge in this drainage area have to be improved because it is in a very deplorable state.

3. <u>Sommelsdijkse Kreek</u>

Discharge to the Suriname River has to be improved on a short-time basis. The creek has to be constructed in accordance with the ultimate discharge criteria. Certain upstream portions have to be improved as soon as possible.

4. Overeemkanaal, Zonnebloemkanaal, and Cottica (Flora) Canal

The improvement of these canals is urgent. The improvement of the water quality of these canals by treatment is necessary for hygienic reasons.

5. Calcutta Kanaal

A bad discharge resulting in poor hygienic conditions is presently existing. A well designed sewer system has to be planned for this area.

6. Knuffelsgracht

Enclosing of the Picorni and Viotte Creeks is very much desired.

7. Steenbakkersgracht

Enclosing is desired.

8. In the remaining areas the urgency is less acute due to the lesser population density.

The above-mentioned list of projects is given in the order of degree of urgency. We would like to mention the following differences between the grades of urgency.

Point 1 is certainly the most urgent; points 2 through 5 are all very urgent; the remaining points are less urgent.

Future studies will indicate in accordance with which principles the improvement program is to be realized. At the same time a cost price calculation for all the construction projects will be estimated.

In addition to the above study, the consulting engineers proposed a sanitary sewage treatment plant on the Flora Canal for which a preliminary design was provided to the Government. This advanced secondary treatment plant was designed for an initial population of 10,000 persons with expansion to 30,000 suggested by the consultants. Industrial waste treatment was not included in the plant capability.

The Government is considering its own alternate design for the Flora Project at present. This design would utilize oxidation ditches with sludge disposal.

The UNDP Preparatory Assistance Mission report recommended that the urban areas contiguous to Paramaribo which were to be included in the detailed designs included in the UNDP(SF) Project would be "only those sewerage systems that will be directly connected to the projected Paramaribo city system (now under investigation by the Government consulting engineers)." It appears that this assumption was optimistic since there are no plans or designs (in preliminary or final stages) for the Paramaribo municipal system nor will any be forthcoming during the life of the UNDP(SF) Project. The unfortunate result of this situation is that the suburban systems will have to be designed independent of any potential connections to a larger urban system.

The only community system in Surinam which collects and treats domestic waste through a piped collection system and a secondary treatment (activated sludge) plant is the small "Via Bella" system, which is operated in a small residential subdivision in Paramaribo by Suralco. This system handles the waste water of approximately 50 houses.

At the Santo Boma prison, septic tank effluent is collected and ponded in a series of lagoons which provide approximately 90 days retention (see Figure IV-15). After post-chlorination the effluent is disposed of in open drainage ditches. A total population of approximately 500 inmates and personnel is served by this system, which is unique in Surinam.

PRESENT METHODS OF DOMESTIC WASTE DISPOSAL

As stated previously, the sanitary sewage treatment in most of the populated areas is effected through the use of septic tanks and effluent collection. In those areas where no piped system exists, the septic tank effluent is discharged into seepage areas or into drainage ditches. During dry seasons, with no storm flows, the total discharge of some of these ditches is partially treated or untreated sewage.

Ditch systems are generally installed at flat grades and at only a slightly lower elevation than the surrounding terrain. Installation of culverts is not controlled as far as invert elevations are concerned. This, together with debris accumulation and encroachment, creates conditions in which sewage collects and stagnates, frequently becoming septic.

Conversely, during the rainy seasons, the overflows which result from the lack of hydraulic capacity spread heavily polluted water. This is further aggravated by tide-locking of the channels at certain periods.

Obviously, the above conditions create severe health hazards and point up the need for immediate consideration of improvements to the sewer and storm drain systems in order to remove sewage and storm waters from the populated areas.

There is less urgency with regard to sewage treatment. The condition of the Lower Suriname River is such that the disposal of untreated sewage into it in the immediate future should not create hazards to public health.

In the more rural areas and in some built-up areas, pit latrines are used almost exclusively, except when no facilities at all are provided. Occasionally a septic tank is installed, particularly when a larger residence is constructed. In these cases, the installations also generally include seepage areas or drain fields.

Little regard is given to the sanitary requirements of the latrines. The World Health Organization, in its Monograph Series No. 39, "Excreta Disposal for Rural Areas and Small Communities," by Wagner and Lanoix, recommends construction of pit latrines in rural areas, but only if they satisfy the following seven requirements:

a. The surface soil should not be contaminated.

b. There should be no contamination of ground water that may enter springs or wells.

c. There should be no contamination of surface water.

d. Excreta should not be accessible to flies or animals.

e. There should be no handling of fresh excreta; or, when this is unavoidable, it should be kept to a strict minimum.

f. There should be freedom from odors or unsightly conditions.

g. The method used should be simple and inexpensive in construction and operation.

Cleaning of latrines is a necessity if the size is insufficient to develop decomposition of excreta. The Ministry of Health is understandably concerned over this practice, particularly since the operation requires workers to break up solid fecal material by hand followed by water-mixing and pumping.

In addition to the unsanitary and socially degrading practices connected with pumping latrines in the country, the depth of the pit and the high elevation of the ground water table combine to cause contamination of the ground water. Further, there is little control over the locations of the latrines, especially on small lots, and the inevitable result is contamination of shallow wells.

PRESENT METHODS OF INDUSTRIAL WASTE DISPOSAL

Methods of disposal of wastes generated by industries in the Lower Suriname River Basin were considered by a short-term consultant. The consultant's report is included in Volume II of this report.

COMBINED AND SEPARATE SYSTEMS

In a country such as Surinam in which the existing major sewerage system is a combined system and in which much of the "system" consists of open ditches, advocating a separate system for sanitary sewage may not be simple.

As already stated, the Paramaribo system has been constructed on the combined principle. Although this is a fairly old system and may contain some design defects, these are not sufficient reasons to abandon or extensively modify the system. The system appears to have been well constructed and functions effectively except in periods of heavy, protracted rainfall.

However, it cannot be justified to design combined sewers for new installations. In Surinam, the high rainfall intensities produce an unbalanced ratio of storm water quantities to sanitary sewage quantities. This large ratio induces the requirement for collection lines which have to operate under a wide range of flow conditions. In order to develop self-cleaning velocities under dry weather flow, the conduits must be installed at steep grades; and this, in turn, results in the need for pumping large quantities of storm water. If pumping economy is to be effected through designing for maximum flow conditions with gravity flows predominant, sewer maintenance problems multiply during long dry seasons.

Buried pipelines for handling huge quantities of storm water are expensive to construct. However, sanitary sewage must be conveyed in this type of system in order to maintain reasonable levels of public health. This, too, would emphasize the need for separate systems.

Treatment of sanitary sewage in municipal plants will become necessary at some future date. Designs of such plants are based not only upon the "strength" of sewage but also upon the hydraulic loading. Plant design and construction in Surinam based upon handling moderate sewage flows during dry seasons and massive flows during rainy seasons is prohibitive in terms of cost and questionable in terms of sound engineering judgement.

CHAPTER 9

BASES FOR DESIGN OF SEWAGE DISPOSAL SYSTEMS

INTRODUCTION

The provision of piped water supplies immediately introduces the requirements for the sanitary removal of the waste water and sewage. In municipalities where a collection and treatment system for sewage is available, connection to the sewer is a minor problem. Where central treatment is not provided or anticipated, special consideration must be given to the proposed method of collection, removal, treatment, and disposal of sewage.

In Surinam, in the immediate future, except in the Paramaribo central area, each type of community will suggest different approaches to sewage handling and treatment. It would appear that much of the treatment will be limited to individual privies, individual or "cluster" septic tank disposal in the rural populated areas, small collection systems with lagooning in those areas where the population concentration justifies this type of treatment, and possibly a few secondary treatment facilities in the built-up areas. The most practical method of treatment of wastes from some of the Paramaribo metropolitan peripheral areas would be in central plants.

One of the basic requirements in the planning of any sewerage system is that of providing a safe, effective, and innocuous disposal method for the wastes collected. It is the primary purpose of this section of the report to anticipate and to assess the various factors governing the disposal of the waste effluents of the project area; to establish, in general terms and in the context of present knowledge, reasonable effluent quality criteria for the various waste discharges; and, finally, to determine what degree of treatment will be necessary to meet these criteria.

The design of treatment facilities must be such as to meet the criteria mentioned previously. For this reason, the method of treatment selected must not only maintain the desired quality of the receiving water, but will also depend upon the character of the waste to be treated. In some countries, discharge requirements are developed when requests are made by prospective dischargers into receiving water or when an investigation of an existing waste discharge following treatment shows the existing requirements to be inadequate.

The discharge requirements which determine the level of treatment generally limit the amount of BOD, grease, suspended matter, and coliform organisms that may be contained in the waste discharge, as well as a minimum amount of dissolved oxygen to be found in the discharge. In addition, depending upon the degree of development in the country, the requirements may include maximum amounts for phosphates, nitrates, phenols, arsenic, color, pH, or any other material or waste characteristic that may have a harmful effect on the receiving water quality.

After the criteria or requirements have been determined, the decision on the waste treatment necessary to meet these criteria must be made. If the area is sewered, existing waste flows will be measured, sampled, and analyzed for BOD, pH, grease, suspended matter, and any other quantity that might influence the selection of the treatment process. If the area is not sewered, other steps are necessary, and the ingenuity and judgement of the designer becomes extremely important.

FACTORS INFLUENCING WASTE DISCHARGE CRITERIA

In developing appropriate discharge criteria, the following basic factors will be taken into account:

a. Beneficial uses made of the receiving waters are the controlling factors in determining water-quality levels that are to be maintained.

b. For every beneficial use, there are certain water-quality requirements which must be met to assure that the water will be suitable for that beneficial use.

c. Receiving waters, which may be canals, rivers, or oceans, have varying degrees of waste assimilative capacity, and the addition of waste materials may change the chemical, physical, and biological characteristics of the water without necessarily creating significant adverse effects on the beneficial water uses.

d. The formulation of waste discharge requirements should be so designed as a) to secure that degree of care in the planning and operation of works for the treatment and disposal of sewage and industrial wastes as will adequately protect the public health and other beneficial uses of waters, and b) at the same time to permit the legitimate planned usage of those waters for receiving suitably prepared wastes so that an orderly growth and expansion of cities and industries may be possible.

Presently, the lower Suriname River and, to a lesser extent, the lower Commewijne and Saramacca Rivers serve as the receiving bodies for the water-borne wastes of the lower Suriname River basin, and it is anticipated that in the future the Suriname, Commewijne, and Saramacca Rivers will continue to serve as the primary means for dispersion and assimilation of the wastes.

Although the lower Suriname River is not utilized as a source of water for public use, it is the major navigation facility of the basin area; hence, it is vital to the growth and overall economy of this area. Also, since some use is made of tributary local canal water for potable water needs by the public, it is essential that waste discharge criteria be so formulated that protection of this vital resource for all planned uses, particularly navigation and waste disposal, is guaranteed.

It should be noted that the discharge requirements to be developed must be based on a number of broad assumptions. It must be recognized that in the future there will be changes not only in the magnitude and variety of new substances which will be discharged but also in the qualitative and quantitative character of the receiving bodies. Because of these factors, the status of waste disposal within the Lower Suriname River area is in a state of flux and must be reviewed, from time to time, in the light of changed conditions and increased knowledge of the many problems involved.

As stated earlier in this report, some investigations were conducted by the Hydraulics Research Division of the Ministry of Public Works, particularly in connection with BOD, solids, chemical characteristics, and dissolved oxygen in the Suriname River and in some of the canals in the lower Suriname River basin.

WASTE DISPOSAL CONSIDERATIONS FOR METROPOLITAN PARAMARIBO

Although consideration of sewerage for the central Paramaribo area is not included in the Project activity plan, this brief reference is included for background purposes.

The receiving waters into which the waste water effluents of Paramaribo are discharged include the Suriname River or the canal systems flowing to final discharge in the river.

The primary factors which must be considered in the disposal of wastes by dilution in surface waters are as follows:

a. The quality level which must be maintained to protect the various beneficial uses of the receiving waters, such as possible drinking water supplies, industrial water supply, and agricultural uses.

b. The volume and strength of the wastes to be discharged.

c. The capacity of the receiving waters to handle the wastes, which in turn is based on the quality of the receiving water available for diluting and absorbing the waste effluents.

At present or during the interim design period, disposal of Paramaribo wastes will have little effect on the dilution capability of the Suriname River.

Quantity of Wastes

A breakdown of the quantities of waste water effluents to be discharged in the Suriname River (including its tributaries) is presented in the table below, based on the assumption that the entire population of Paramaribo is to be sewered (both at interim and for ultimate conditions). The table includes rough estimates of the population tributary to the various disposal points, as well as a very general breakdown of domestic and industrial wastes for the interim and ultimate conditions.

Estimated Quantities of Waste to be Discharged into the Suriname River

	Tributary Population	Domestic Flow m^3/day	Industrial Flow m^3/day	Total Flow m^3/day
Interim	400,000	60,000	15,000	75,000
Ultimate	700,000	118,000	28,000	140,000

Notes: a) Interim flows are based on the 1987 estimated population figures and water use.

b) Ultimate flows are based on 2001 conditions.

Waste Characteristics

The waste waters of the Paramaribo area will be derived from two basic sources, domestic sewage and industrial wastes.

Domestic Sewage: In addition to the discussion of the characteristics of domestic sewage presented in the section of this report entitled "Bases for Design" are the following remarks.

Besides adding organic waste materials, domestic use of water will add dissolved mineral salts. It is estimated that domestic water use will increase the total dissolved solids (TDS) content by about 250 to 300 mg/l. This represents an insignificant contribution to the Suriname River solids content. The table below presents a general list obtained from WHO sources of the additions to various major chemical constituents found in water through domestic use. Domestic wastes, of course, may also contain biological pathogens including bacteria and viruses.

Constituent Additions After Domestic Water Use

Constituent	Added Amounts (mg/1)
Total dissolved solids	250 to 300
Boron	0.1 to 0.4
Sodium	40 to 70
Potassium	7 to 15
Magnesium (as $CaCO_3$)	15 to 40
Calcium (as $CaCO_3$)	15 to 40
Total nitrogen	20 to 40
Sulfate	15 to 30
Phosphate	20 to 40
Chloride	20 to 50
Alkalinity (as $CaCO_3$)	100 to 150

Industrial Wastes: Industrial waste matters of varying kinds are contained in the spent process waters of manufacturing establishments and are produced in washing, flushing, extracting, and impregnating operations. Like domestic sewage, industrial wastes contain suspended, colloidal, and dissolved solids of mineral and organic origin. Pathogenic organisms are rarely found in industrial waste waters.

The strength of industrial waste waters varies widely from industry to industry and with changing manufacturing procedures within the same industry. Hence, it is not feasible to attempt to assign definite values to the characteristics of industrial wastes to be produced in Paramaribo, until a comprehensive review of all major industrial wastes is performed.

Effective control of the discharges of industrial wastes into the sewerage system is best accomplished by regulations through which existing and proposed discharges may be reviewed in order to evaluate their effects, and by which reasonable pretreatment can be required if needed. Such industrial waste regulations should be based on the philosophy that the sewerage should afford maximum service for industry, and hence should receive and handle wastes or treated wastes to the extent possible without impairment of the sewerage system's capacities for receiving and handling domestic wastes. On this basis a sample of typical regulations submitted by the WHO short-term consultant for industrial wastes has been supplied to the Government of Surinam.

WASTE WATER RECLAMATION

At the writing of the report, no consideration had been given to studying waste water reclamation. In some countries in which chronic shortages of water for community supplies or for agricultural purposes are experienced, the practice of waste water reclamation is pursued in order to meet these demands. However, this frequently implies the need for additional investment in treatment elements, an approach which would be difficult to implement in Surinam, where the needs for agriculture are not on the same scale as the discharge which would be available from treatment plants. Depending upon public contact with the irrigated area, or with the produce grown in the irrigated area, more or less rigorous disinfection criteria would need to be applied.

Some standards proposed by the California Department of Health (USA) are:

	Max. MPN
For irrigation of parks, playgrounds, etc.	5/100 ml
For irrigation of golf courses	23/100 ml
For vegetable and raw foods	0/100 ml
For certain other crops	Partial disinfection

Further, if it is desired to reuse effluent for drinking water, public health standards and esthetics would practically demand tertiary treatment. The tertiary treatment chosen would have to be justified by an economic analysis. In other words, the reclaimed or tertiary treated effluent would have to cost less than the cost of hauling fresh water or treating raw water. This, in turn, would require a comparative investigation of the costs of desalination of water of extremely high chloride content.

GENERAL SEWERAGE DESIGN

Introduction

The design of collection, treatment, and disposal facilities for waste water is primarily controlled by the estimated BOD and hydraulic loadings to be handled, as well as the method of collection and treatment which is justified. In developing cost estimates, it is necessary to determine on a unit basis not only the flow or volume of waste but also its strength and composition. With respect to flow in collection systems, peak flow rates must be known in order to determine the required hydraulic capacities of sewers, pumping stations, treatment plants, and effluent disposal facilities. Similarly, strength and composition must be known in order to determine the degree and type of treatment required to produce an effluent of acceptable quality. Thus, a study of waste volumes and characteristics is a necessary preliminary step to the development of design criteria.

Waste Water Quantities

There are three important components of waste water volume: the first consists of sanitary sewage; the second, of industrial waste; and the third consists both of subsurface water, which enters a sewer system through joints and other openings, and of storm water, which enters through manhole openings and illicit drain connections. The third component is generally termed infiltration.

Because the major portion of infiltration occurs during periods of heavy rainfall, it is necessary to consider sewage volume in terms of both dry weather and wet weather flow. Dry weather flow consists of waste water occurring during periods of zero or minimal rainfall. Similarly, wet weather flow consists of waste water plus infiltration and storm water inflow. Dry weather flow usually determines the normal loading to be imposed on major units of a treatment plant, whereas wet weather flow indicates the hydraulic capacity required for sewers, pumping stations, and force mains.

Domestic Sewage

Unit or per capita sewage flows are conventionally determined from a study of recorded flow data and population served. Then, with the basic data of present flow at hand, future changes are projected and suitable future design allowances are determined.

In Surinam, as previously stated, only one fairly extensive sewerage system is presently in operation: the Paramaribo system, which is designed on the combined basis. Because the average per capita water supply (hence per capita sewage flow) is restricted by limitations in the present water supply system, metering of water flows within the Paramaribo system does not provide a reliable guide to the per capita waste water flows of the population served. In view of the above factors, and the necessity for establishing unit sewage flows for use in the design of separate sewerage systems, per capita sewage flows have been determined by estimation. For design purposes, therefore, the quantity of sewage obtained from residential areas is estimated to be 80% of the established per capita water allowances (as noted in Part III). This percentage generally corresponds to conventional practice for metropolitan areas of this size.

Industrial Waste

Because of the varied nature (type and size) of the existing anticipated future industries within Surinam, industrial waste flows are difficult to predict with any degree of accuracy. Hence, only general estimates of the industrial contribution are possible. Certain types of industrial wastes will require pretreatment prior to being discharged into the public sewers, in order to provide proper protection to the sewerage system. The extent of such pretreatment will be indicated in the suggested ordinance included as Annex I.

It is expected that 50% of the industrial water consumption can be estimated as being discharged to the sewerage system. This value has been found in general to be a reasonable allowance for industrial areas where suitable regulatory controls are applied.

In areas served by combined sewers, it is estimated that 80% of the industrial water consumption is discharged to the public sewers.

Infiltration

In areas served by combined sewers, no consideration has been given to infiltration because the quantities involved are generally insignificant in relation to total hydraulic capacities of the sewers.

For areas with separate sewers, an infiltration allowance of metric equivalent to 1,900 liters per centimeter of diameter of sewer per kilometer per day will be assumed, corresponding to general practice elsewhere where ground water is prevalent and sewers reasonably well constructed. The purpose of this procedure will be to obtain a reasonable estimate of infiltration on a per acre basis which can properly be used for estimating total infiltration in the future.

Design Flows

The design of sewers is based on maximum rather than average flow, and the ratio of maximum to average flow varies according to location within the sewerage system—that is, on the extent of the tributary from which sewage is being collected. For design purposes, estimates will be made of this peaking factor (rate of maximum to average flow), but is tentatively assumed as a factor of 2.0.

Basis of Design for Sewerage Systems in Rural Areas

Population Growth: The average growth in population for areas to be served by sewerage has been assumed at approximately 4% per year.

Additional assumptions were covered in this report in the sections on water system design.

Rate of Sewage Flows: The number of persons in a home in areas to be served is assumed to be six. As stated earlier, flow can only be predicted, since no existing sanitary sewer systems are presently in service.

Average daily flow:	70 l/person
Peak daily flow :	140 l/person

This includes effluent from sinks, toilets, bathrooms, and so on.

Peak Sewage Flows From Specific Buildings - Movie Houses: Average capacity 1,000 persons, with three performances on Saturdays, each lasting two hours. All other days two performances per day.

- For septic tanks and drain field design:

Daily : 2.4 m^3
Hourly: 0.4 m^3

- For overall treatment and gravity system design:

Daily : 3.0 m^3
Hourly: 0.5 m^3

Restaurants: Average capacity 35 tables, four persons per table. Greatest usage from 12:00 noon on Saturdays to 1:00 Sunday morning. Also included is kitchen cleaning effluent.

- For septic tank and drain field design:

Daily : 0.6 m^3
Hourly: 0.1 m^3

- For overall treatment and gravity system design:

Daily : 4.8 m^3
Hourly: 0.5 m^3

Local Shops: Average 100 persons passing through every hour at peak time. Peak period lasts four hours per day.

- For septic tank and drain field design:

Daily : 0.5 m^3
Hourly: 0.1 m^3

- For overall treatment and gravity system design:

Daily : 0.6 m^3
Hourly: 0.2 m^3

Schools: Flow based on 12 grade-school and four preschool classes:

- For septic tank and drain field design:

Daily : 3.0 m^3
Hourly: 0.4 m^3

- For overall treatment and gravity system design:

Daily : 0.4 m^3
Hourly: 0.5 m^3

Service Stations: Stations are open 16 hours per day. Included are car washing outflows (only for overall system design).

- For septic tank and drain field design:

Daily : 0.8 m^3
Hourly: 0.1 m^3

- For overall treatment and system design:

Daily : 20.0 m^3
Hourly: 1.2 m^3

Government Offices: Utilized for local administration tasks. Forty employees per eight-hour day.

- For septic tanks and drain field design:

Daily : 0.7 m^3
Hourly: 0.1 m^3

- For overall treatment and gravity system design:

Daily : 1.0 m^3
Hourly: 1.15 m^3

Recreation Buildings: Capacity 200 persons on Saturday, and building is used for a six-hour period. Also includes effluent from kitchen facilities, except for septic tank design:

- For septic tank and drain field design:

Daily : 8.0 m^3
Hourly: 1.0 m^3

- For overall treatment and gravity system design:

Daily : 9.2 m^3
Hourly: 1.5 m^3

Medical Buildings (polikliniek): Capacity 50 visitors per day and five professional staff. Buildings open for eight hours per day.

- For septic tank and drain field design:

Daily : 0.5 m^3
Hourly: 0.1 m^3

- For overall treatment and gravity system design:

Daily : 0.6 m^3
Hourly: 0.15 m^3

Medical Building and Small Hospital: Hospital with a capacity of 20 beds and kitchen facilities. Included are seven employees, two employees each on a three-shift program.

- For septic tank and drain field design:

Daily : 1.9 m^3
Hourly: 0.2 m^3

- For overall treatment and gravity system design:

Daily : 2.4 m^3
Hourly: 1.0 m^3

Shop for Maintenance Equipment: Flows based on a shop which serves an agricultural community of approximately 1,000 persons. Includes wash rack effluent only for overall treatment and gravity system design. Shop open eight hours per day.

- For septic tank and drain field design:

Daily : 0.5 m^3
Hourly: 0.1 m^3

- For overall treatment and gravity system design:

Daily : 5.1 m^3
Hourly: 0.6 m^3

Note: All above-mentioned sewage flows are only for specific purposes and should only apply to the rural areas located outside the metropolitan area of Paramaribo.

Collection System: Minimum size of pipe used is 6".

Sewage Characteristics

Sewage characteristics are significant in that they determine both the type and the degree of treatment which must be provided to meet established discharge criteria, and also may affect the design of sewers and other sewerage facilities.

In a comprehensive study of this type, data obtained from representative samples collected at various existing facilities are analyzed to determine present waste water characteristics. After the present character of the waste water is established, this information will be used as a basis for estimating future design loadings. Some BOD sampling has been made by the Government, but this reflects only the receiving water in the Suriname River. Sampling and analyses for evaluating per capita BOD have been performed in Moengo by project staff.

Preliminary Design Loadings

The waste water of the project area will be derived from predominantly domestic sewage and smaller industrial discharges. Since industries of various types could be located throughout the project area, it is hardly possible to estimate the exact magnitude and nature of the industrial loadings. However, it is assumed that industrial loadings will be regulated to comply with the provisions contained in suitable future industrial waste ordinances. The design factors adopted for various stages of development are presented below:

Design Factors for Waste Characteristics

Factor	1987 Value	
	lbs/cap/day	g/cap/day
BOD, 5 day 20°C	0.17	78
Suspended solids	0.20	90

The above design concentrations of BOD and suspended solids estimated for the year 1987 are probably higher than existing values to allow for the increases in living standards and in industrial loads on the municipal systems.

DESIGN OF COLLECTION SYSTEMS

The design criteria used in the general design of sewerage projects are presented in the following sections.

Sewers

Roughness Coefficients: The Manning formula has been used for the solution of problems involving open-channel flow in sewers. The following values of "n" (roughness coefficient) have been adopted for use with this formula, corresponding to field observations of existing sewers and anticipated future conditions.

Design "n" values: 0.008 for all proposed sewers of 18-inch and smaller diameter—flow at half depth.

$$\text{The Manning formula:} \quad V = \frac{1486}{17} \times R^{2/3} \times S^{1/2}$$

Roughness coefficient of above-mentioned value is being used for plastic pipe for gravity systems.

Analysis "n" values: 0.015 for existing sewers in average condition, 0.017 for existing sewers in poor condition.

Design Depth of Flow: Sewers of 15-inch diameter and less are designed at half depth of flow. This allows flexibility in providing for additional flows because of future increases in population.

Minimum Slopes and Velocities: For smaller sewers, the following minimum invert slopes have been adopted:

Diameter of Sewer (Inches)	Minimum Invert Slope
6	0.0030
8	0.0022
10	0.0016
12	0.0014
15	0.0010

Minimum slopes are such as to provide a velocity of 1.7 feet per second (fps) with the sewer flowing at design depths, except that higher minimum velocities are adopted where necessary to inhibit the formation of hydrogen sulfide. Where sulfide problems are significant, good practice would suggest average velocities not less than 3 fps and preferably as high as 4 fps.

Manhole Location: Placing of the manholes is based on maintenance procedures: for sewers 6" to 15" in diameter the maximum distance between manholes is 400 ft (122 m). Manholes will be placed at changes of grade, direction, and diameter.

Elevation Difference in Manhole Inverts: The vertical drop across manholes (minimum drop of invert) for small sewers (15" and smaller) with straight-through flow is adopted as 0.10 foot (3 cm). For a side inlet, the minimum drop of invert is 0.20 foot (6 cm). For straight-through flow across manholes with different sizes of pipes with matched water surfaces see the following table:

Drop in Feet
(Diameter of inlet in inches)

Diameter of Outlet	6"		8"		10"		12"	
	Feet	Meters	Feet	Meters	Feet	Meters	Feet	Meters
8"	0.008	(.02)						
10"	0.17	(.05)	.08	(.02)				
12"	0.25	(.08)	.17	(.05)	0.08	(0.02)	0.02	(0.005)
15"	0.38	(.11)	.29	(.09)	0.21	(0.06)	0.13	(0.04)

The maximum drop for straight-through flow is 0.60 ft (0.18) and for side inlet 1.0 ft (0.30 m).

For manhole structures with side inlets (confluence structures) the drop of inverts is calculated using the principle of the conservation of momentum (pressure + momentum).

Lift Stations

Capacity of wet wells and pump stations are designed to handle the average sewage flow. Piping is designed for peak flows; pumps and motors can be replaced and reused to handle future increased flows. Wet wells are designed to handle a flow of five minutes' time between successive starts of a pump (where possible). Enough flexibility is available to accommodate future increased flows which decrease the time lapse to 2 1/2 minutes, set as a minimum between pump starts. Wet wells are poured-in-place reinforced concrete structures. Slab thickness is considerable since the maximum gravity sewer depth would be kept at approximately 5.5 m below the existing ground. Ground water level, as stated earlier, is at an average of approximately 0.50 to 1.00 m below the existing ground level, causing uplift, which has to be carefully examined in station design. Each pump station has a standby pumping capacity equal to the required pump and power capacity to handle the flow. Pumps are of the submersible type driven by electric motors.

Each station operates fully automatically on the sewage level in the wet well. Alarms are provided for high water, malfunction of pumps, and power failures. A standby power plant is provided to operate the pump in case of power failure.

For pump station discharge force mains, underground plastic pipe is used. Above-ground materials for bridge crossings are made of cast-iron pipe. Velocities are kept to a maximum of 10 ft/sec and a minimum of 2.5 ft/sec.

Hydraulic calculations are based on the Hazen-Williams formula with a friction coefficient of C = 120.

WASTE STABILIZATION PONDS

Waste stabilization ponds provide a useful method of waste water treatment and disposal in small communities where both funds and trained personnel are in short supply. In these ponds, beneficial organisms stabilize the waste water into a liquid that can be released to the environment without endangering man directly or affecting the environment adversely.

Due to the uniformly distributed sunshine and warm temperatures prevailing in Surinam, the type of stabilization pond treatment adopted was that of anaerobic ponds in series with facultative ponds and, where required, maturation ponds for reduction of the algal concentration in the effluent.

Basis of Design

1. Anaerobic ponds

   a. Area and loading: The anaerobic ponds shall be designed for a daily loading of 700 kg of BOD per hectare.

   b. Pond shape: The shape of the ponds should be such that there are no narrow or elongated portions, preferably rectangular, with a width-to-length ratio of 1:4. Dikes should be rounded at the corners to minimize accumulations of floating materials.

2. Facultative ponds

a. Area and loading: The facultative ponds shall be designed for a daily loading of 150 kg of BOD per hectare.

b. Pond shape: The recommendations for the anaerobic ponds about shape apply also to facultative ponds, except that the width-to-length ratio should not exceed 1:3.

3. Maturation ponds

Maturation ponds are designed in a similar manner to the facultative ponds but with half of the area.

Pond Location

a. Distance from habitation: A pond site should be as far as practicable from habitation or any area which may be built up within a reasonable future period.

b. Prevailing winds: If practicable, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas.

c. Surface run-off: Location of ponds in watersheds receiving significant amounts of run-off water is discouraged unless adequate provisions are made to divert storm water around the ponds and otherwise protect pond embankments.

Pond Construction Details

1. Embankments and dikes

Embankments and dikes should be constructed of relatively impervious materials and compacted sufficiently to form a stable structure. Vegetation should be removed from the area upon which the embankment is to be placed. Where a high ground water problem exists, the limiting cut in the existing ground shall be kept at a maximum of 1.0 m. Surrounding dikes are built on compacted fill to the required height above the existing ground. The minimum embankment top width should be 2.50 m to permit access of construction and maintenance vehicles.

Embankments shall be seeded above the water level. A perennial type, low growing, spreading grass which withstands erosion and which can be kept mowed is most satisfactory for seeding of embankments.

a. Slopes - Embankment slopes should be:

- Inner: three horizontal to one vertical

- Outer: three horizontal to one vertical

b. Freeboard - Minimum freeboard shall be 50 cm.

c. Depths - Normal operating depths shall be as follows:

- Anaerobic pond = 1.0 m
- Facultative pond = 1.20 m
- Maturation pond = 1.20 m

2. Pond bottom

a. Uniformity: The pond bottom should be as level as possible at all points. Finished bottom elevations should not be more than 7.0 cm from the average elevation of the bottom.

b. Vegetation: The bottom should be cleared of vegetation and debris. Organic material thus removed should not be used in the dike construction.

c. Soil formation: The soil formation or structure of the bottom should be relatively tight to avoid excessive liquid loss by percolation or seepage. Soil borings and tests to determine the characteristics of surface soil and subsoil should be part of the preliminary surveys to select pond sites. Gravel and limestone areas must be avoided.

d. Percolation: The ability to maintain a satisfactory water level in the ponds is one of the most important aspects of design. Removal of porous topsoil and proper compaction of subsoil improve the water-holding characteristics of the bottom. Removal of porous areas, gravel, or sand pockets and replacement with well-compacted clay or other suitable material may be indicated.

3. Influent lines

Influent lines should be located along the bottom of the pond so that the invert of the pipe is just below the average elevation of the pond bottom. This line can be placed at zero slope. The influent line to a single-celled pond, or the first pond in multi-celled ponds operating in series, should be essentially center-discharging. Horizontal inlets should be used for gravity flow.

Either vertically upward or horizontally-discharging influent lines may be used where the sewage is pumped to the pond. The end of the discharge line should rest on a suitable concrete apron with a minimum size of 60 cm^2.

4. Overflow structures and interconnecting piping

Overflow structures should consist of a manhole or box equipped with multivalved pond draw-off lines or an adjustable overflow device so that the liquid level of the pond can be adjusted to permit operation at depths of 60 cm to 1.20 m. The lowest of the draw-off lines to such structures should be 30 cm off the bottom to control eroding velocities and avoid pickup of bottom deposits. The overflow from the pond should be taken near, but below, the water surface to release the best effluent and insure retention of floating solids.

When possible, the outlet structure should be located on the windward side to prevent short-circuiting.

Interconnecting piping for multiple unit installations operating in series should discharge horizontally near the secondary pond bottom to minimize the need for erosion control measures and should be located as near the dividing dike as construction permits.

5. Miscellaneous

The pond area shall be enclosed with a suitable fence to exclude livestock and discourage trespassing. All access gates should be provided with locks.

Appropriate signs should be provided along the fence to designate the nature of the facility and advise against trespassing.

Provisions for flow measurement should be provided. Facilities for the installation of inlet and outlet weirs should be adequate for most installations. If required, the final effluent should be chlorinated before being discharged into a drainage ditch or watercourse.

SEPTIC TANK DESIGN

The common method for sewage treatment and disposal, generally in rural areas, including those of "developed" countries, consists of an adequate septic tank for the settling and treatment of the sewage together with a subsurface leaching system for the disposal of the overflow. In Paramaribo, as previously stated, the effluent is collected in the combined system and disposed of in the Suriname River.

In the coastal area, specifically in that part of the project area for which the Supply Group I systems are planned, the relatively high water table elevations (40 to 80 cm below surface level, generally) generates problems in the design of leaching fields or drainage areas. The design of leaching fields usually is based on the ability of the soil from a depth of one meter up to three meters (and in some cases up to six or eight meters when permeable strata are deep) to absorb water.

A number of factors, in addition to the ground water table location, affect the "treatment" of septic tank effluent. The soil structure, earthworm presence, and root penetration have a definite effect on the aerobic oxidizing bacteria and animal organisms, liquids, or gases. A waterlogged soil tends to destroy these effective organisms and, in turn, preserve the organic matter in the effluent. The use of well-drained, artificial leaching fields may be used in lieu of a conventional leaching system, but this generally results in disposal of the effluent into ditches, canals, or other water sources.

Septic Tank Design Characteristics

For design purposes, a number of six persons for each house is used, which is an average and a relatively accurate figure for these rural areas. Project staff have conducted several house counts in order to verify this figure. Only toilet effluent is considered.

1 home
(6 persons) - septic tank dimensions:

1.00 m inside diameter with a depth of 1.00 m, drain field 3.70 m x 2.00 m, depth 1.00 m
4 perforated rows of 4" pipe
6" pipe for the distribution structure
6" effluent pipe - 4" diameter collection system

2 and 3 homes
(12-18 persons) - cluster type septic tank dimensions:

1.00 m inside diameter with a depth of 2.00 m, drain field max. 6.00 m x 2.00 m, depth 1.00 m
4 rows of perforated 4" pipe
6" pipe for the distribution structure
6" effluent pipe - 4" collection system

(A graph of septic tank volumetric requirements is attached as Figure IV-16.)

SANITARY PRIVIES

Where people cannot afford the construction of a septic tank system, the sanitary privy is the means of disposal of human excreta.

Due to their significance in the protection of health in the rural areas, the construction of privies must be made an integral part of the general plan for collection and disposal of sanitary wastes.

Privies are composed of two parts: the superstructure and substructure. Privacy and protection against the elements are insured by the superstructure, and the sanitary requirements by the substructure. Both components should be held as simple as possible to meet the economic needs of the population that they must serve.

Of all types of privies the most widely used is the pit privy. With a minimum of attention to location and construction, there will be no soil pollution and no surface or ground water contamination. The excreta will not be accessible to flies if the hole is kept covered, but even when the hole is left open, the fly problem will not be very great since flies are not attracted to dark holes and surfaces. A good superstructure helps to keep the light from the pit. There is no handling of the material; odors are negligible; and feces are normally out of sight. The pit privy is simple in design, easy to use, and does not require operation. The life span will vary from five to 15 years depending upon the capacity of the pit and the use and abuse to which it is put. Its chief advantage is that it can be built cheaply, utilizing indigenous labor.

1. <u>Pit privy</u>

The pit privy consists of a hand-dug hole in the ground covered with either a squatting plate or a floor slab provided with a riser and seat. A superstructure or house is then built around it. The privy is installed on a mound in order to prevent flooding.

The pit is usually round or square for the individual family installation and rectangular for the public latrine. Its dimensions vary from 90 cm in diameter to 1.20 m^2. Common figures for family latrines are 90 cm diameter or 1.0 m^2. The depth is usually about 2.50 m, but may vary from 1.80 to 5.0 m.

It is often necessary to provide a pit lining to prevent the sides from caving in. This is true especially in rainy climates where the privies are dug in fine-grained alluvial soils, sandy soils, or similar formations, or when the pit penetrates deeply into ground water. Materials commonly used for this purpose include brick, stones, concrete blocks, lumber, rough-hewn logs, split cane, and bamboo.

The base serves as a solid, impervious foundation upon which the floor can rest. Properly made of hard durable material, it helps to prevent the entrance of burrowing rodents and of surface water into the pit. The foundation should be at least 10 cm wide on top in order to provide a good surface for the floor to rest upon, and 20 cm at the bottom to give a stable contact with the ground. The base should be tight enough to raise the floor 15 cm above the level of the surrounding ground to protect the pit from possible flooding.

The following materials may be used in the construction of the base:

a. Plain or reinforced precast concrete, utilizing the same mix as for floors

b. Soil cement, 5 to 6% cement mixed with sandy clay soil and tamped at optimum moisture content

c. Tight clay, well tamped at optimum moisture content

d. Brick

e. Stone masonry

f. Rough cut logs, termite resistant hardwood

The floor supports the user and covers the pit. It should be constructed so as to fit tightly on the base, and extend to the superstructure walls. Materials commonly employed include:

a. Reinforced concrete

b. Reinforced concrete with brick filler

The squat type slab for pit privies had been found more suitable for rural conditions. However, in certain cases a slab provided with a riser and seat may be found more acceptable.

All factors considered, appropriate dimensions for the concrete slab may be 1.0 x 1.0 m in overall size. The thickness of slabs varies a great deal in practice. In order to reduce weight, the tendency has been to reduce the thickness to a minimum consistent with safety. In this respect, however, much depends on the quality of the concrete and the reinforcement available. When these factors are favorable, the slab may be of uniform thickness throughout and not less than 6.0 cm thick.

The function of the mound is to protect the pit and base from surface run-off which otherwise might enter and destroy the pit through flooding. It should be built up to the level of the floor and be very well tamped, and should extend 50 cm beyond the base on all sides. In exceptional cases, such as in flood plains and tidal areas, the mound may be built up considerably above the ground for protection against tides and flood waters. The mound is normally built with the earth excavated from the pit.

2. <u>The house or superstructure</u>

The house or superstructure affords privacy and protects the user and the installation from the weather. A properly built superstructure conforms to the following characteristics:

a. Size should be the dimensions of the floor slab

b. Ventilation should be provided through openings 10-15 cm wide at the top of the house wells to facilitate constant ventilation.

c. The roof should cover the house completely and have a large overhang to protect the ground and walls from rain and roof drainage. The height of the roof over the slab near the entrance door should be 2.0 m or more.

d. Natural light should be available. However, the superstructure should provide sufficient shade over an uncovered seat or hole in order not to attract flies. Materials used in the construction of the superstructure include the following, among others:

   - Cut lumber
   - Asbestos cement sheets
   - Sheet metal
   - Palm or grass thatch
   - Brick

3. Location of the pit privy

Regarding the location of privies with respect to sources of water supply, experience has shown that in average soil conditions a privy should be separated from a house well by a distance of not less than 10.0 m.

Regarding the location of privies with respect to dwellings, it has been observed that the distance between the two is an important consideration in the acceptability of the sanitary facilities. A privy will be more readily used and more likely will be kept clean if it is reasonably close to the house which it serves. However, on the other hand, the privy should not be too close to the house. It should preferably be built at a nominal distance of 6.0 m or more from the dwelling.

4. Care and maintenance

The pit privy needs little maintenance, but there are some simple requirements which must be satisfied in order to guarantee acceptance as well as to ensure its lasting for the design period.

The superstructure should be kept clean at all times. When not cleaned regularly it becomes objectionable, with the result that the users will prefer to defecate in the area around the privy or in a neighboring bush. The door and walls should be kept in a good state of repair.

Because the mound has a tendency to be eroded by heavy rains, it should be repaired immediately. For esthetic reasons and to make it more acceptable, the superstructure should be painted a light color or "whitewashed." The immediately surrounding vegetation should be kept trimmed.

DESIGN PERIODS FOR SEWERAGE SYSTEMS

The following design period criteria are provided to allow for an orderly expansion in capacities of sewerage systems.

Land Requirements

Similar to the situation for water supplies, most land is under central government control. Therefore, it should be no problem to acquire ample acreage in the first instance to meet anticipated ultimate requirements (i.e., for a design period of 40 years or more). Inadequate initial acreage can severely inhibit expansion at a later date. Wherever possible, a margin of land around sewage treatment plants should remain permanently undeveloped as a "buffer zone" to prevent early development of residential property.

Outfall Structures, such as Ocean Outfalls

A design period of 40 years or more.

Major Structures, such as Pumping Stations

The design period should be 40 years or more. The mechanical and electrical equipment within pumping stations should be designed for periods of 15 years or less with provisions for later modifications or additions.

Major Sewage Treatment Plant Elements

The design period should be 15 years or less, with provisions for parallel expansions.

Collection Systems

The design period for trunk sewers and secondary sewers should be 40 years or more.

CHAPTER 10

PROPOSED SEWERAGE SYSTEMS

KWATTA-LEIDINGEN SYSTEM

General

The proposed piped sewerage system and the cluster type septic tank system for the Kwatta-Leidingen area are included as Plan IV-19.

The Kwatta-Leidingen system has been investigated on the basis of two alternate methods of collection and disposal: 1) a piped system of collection with treatment and disposal through lagooning and 2) a cluster type septic tank system with small drainage areas.

Any sewerage collection system in the Kwatta-Leidingen area as well as in most of the coastal region of Surinam will be expensive to build and maintain, regardless of the method used. The factors which develop this condition are 1) the extremely high ground water table and 2) the method of road construction in which native material is scraped from both sides of the road right-of-way to build the fill for the road-bed, thus creating wide irrigation and drainage channels along each side of the right-of-way. House services must cross these channels, either on supports or by use of siphons, or parallel lines must be constructed. Another effect of this method of road/dike construction is to develop a height of crown in the road which is considerably above the first floor elevation of houses adjoining the roadway, thereby impelling the need for deep excavation in the roadway for gravity sewer lines.

Location, Boundaries, and Description of Area

The Kwatta-Leidingen sewerage project is located in the same area previously described to be served by the Kwatta-Leidingen water supply system. The area is characteristic of the rural areas contiguous to Paramaribo, with the typical density of housing.

The boundaries of the sewerage zone encompass a somewhat larger service area than was included in the water supply project, and this was based on the assumption that piped water would be available in the southeasterly section through another system of supply.

Population Served

The design population for the sewer service area was based on the similar design population for the water supply system and this, in turn, was projected from an actual present day house count. For estimating and design purposes the total population was projected to 1987 and is estimated at 25,000.

Design Analyses

Piped collection system

As stated earlier, the average flow was estimated at 80 liters per capita per day, and this includes domestic, commercial, and industrial contributions. A peaking factor of 2.0 is utilized for the collection system design.

This would develop an average daily flow of 2 million lpd and a peak flow of 4 million lpd. Infiltration, based on an approximate value of 1,900 liters per day per centimeter of diameter per kilometer of pipe and averaged over the total collection system represents approximately 10% of the peak flow. Design criteria for sewerage design were discussed in an earlier section.

Collection system design was based on establishing initial depths of invert at 1.0 meter and installing pumping stations when a 6.5-meter depth was reached.

Following are the pipe sizes and lengths included in the collection system. Pipe used would be vitrified clay, asbestos-cement, or plastic (PVC or high density polyethylene).

6-in diameter	21,200 m
8-in diameter	39,100 m
10-in diameter	6,200 m
12-in diameter	3,000 m
15-in diameter	2,500 m

Because of the flat terrain, a total of 19 pumping stations will be required. It is proposed to use package-type lift stations complete with two self-priming, horizontal centrifugal pumps; valves; internal piping; motor control center; and liquid level control system. The pumps would be required to handle solids up to four inches in diameter.

The pumping stations will be above ground with wet wells installed as shown on Plan IV-19.

Also shown on the above drawing are the location and details of the sewage lagoon system for the sewerage projects. Immediate needs would compel the initial construction of two lagoons of approximately 100 x 300 m size, in order to provide 90-day retention of the sewage. At an intermediate phase, during 1978, a third lagoon would be needed and a fourth would be built by 1987. Discharge, after chlorination, would be into canals.

Cost Estimates for Kwatta-Leidingen Systems

Piped sewerage system

Collection system	Sf. 3,604,000
Lift stations	190,000
Stabilization ponds	317,000
	Sf. 4,111,000

Cluster septic tanks

Piping	Sf. 747,000
Septic tanks	226,000
Filters	527,000
	Sf. 1,500,000

Cluster Septic Tank System

The design of the septic tank system (see Plan IV-20) was based on serving either one, two, three, four, or five houses in a cluster. The limits are reached when the limit of gravity flow is reached, and the situation in the Kwatta-Leidingen area is such that the maximum number of houses which can conceivably be served is five, because of the flat terrain.

Again, a flow of 80 l/c/d was used. Detention time in the septic tank was assumed to be 12 hours. Because of the high water table and the waterlogged condition of the existing soil, it is proposed to include an open sand filter with each septic tank. Design of the filters is based on a loading rate of 100,000 gallons per acre/day. After filtration the effluent will be discharged into canals.

The septic tank construction is to be in two stages with the final stage to include the construction of an open sand filter for each septic tank. The design of the filters is based on a loading rate of 100,000 gallons per acre/day. As the effluent from the septic tanks will be discharged into canals, the filter construction will be required as soon as the water in the existing canals is improved to a higher quality than now exists.

NIEUW NICKERIE SYSTEM

General

The drawings for the proposed Nieuw Nickerie sewerage system are included as Plans IV-21 to -34.

Nieuw Nickerie has a piped water supply system which is operated by the Surinam Water Company.

At the present, sewage is discharged into the ditches of the storm drainage system. Sullage water is discharged directly, and flush toilet effluent is discharged after treatment in septic tanks.

The open canals create a problem in that most are hydraulically inefficient and subject to rapid weed growth; because of this, together with trash dumping into the canals, serious sanitation problems are created. During the rainy season the canals overflow as a result of the lack of hydraulic capacity, spreading heavily polluted water. This situation is further aggravated by occasional tide locking of the channels. Obviously, the above conditions create severe health hazards and point up the need for immediate consideration of improvements to the sewer and storm drain systems. A separate sewage collection system will be expensive to build and to maintain regardless of the method used. Two factors create this condition: 1) the drainage ditches running on both sides of the streets and 2) the height of the crown of the streets that are considerably above the first floor elevation of the houses fronting the street, thereby impelling the need for deep excavation in the roadway for sewer lines.

Nieuw Nickerie is located on the east bank of the Nickerie River near its mouth in the Atlantic Ocean. The topography of the region is typical of the coastal plain presenting a flat swampy terrain. The soil consists of a top layer of impervious sandy clay 30 to 40 m thick with sand underneath. The boundaries of the sewerage zone encompass the area served by the water supply system.

Population Served

The design population was estimated on the basis of the saturation of the area within the city limits. A population of 16,000 was obtained by this method, and it corresponds to the projection to 1976 of the present population of 11,000 at the growth rate of the last 10 years. After 1976 the excess population will be taken care of by the development of a new area adjacent to the present city limits. This development is now under study by the Planning Office of the Ministry of Public Works.

Design Analysis

Collection system

Based on data furnished by the Surinam Water Company on water consumption in Nieuw Nickerie, the average design flow was estimated at 150 liters per capita per day; this includes domestic, commercial, and industrial contributions. A peaking factor of 2.5 was adopted for the collection system design. From this will result in an average daily flow of

2,400 m^3 or 28.7 l/sec and a peak flow of 70 l/sec. Infiltration, based on an approximate value of 1,900 liters per centimeter of diameter per kilometer of pipe and averaged over the total collection system represents approximately 10% of the peak flow.

Design criteria for sewerage design were discussed in an earlier section.

In the design of the collection system an initial depth of 1.50 m was used in order to assure that the crown of the sanitary sewers would be at a lower elevation than the invert of the ditches of the storm drainage system. Pumping stations were installed when a depth of 5.50 m was reached.

Following are the pipe sizes and lengths included in the collection system force main and outfall. Pipe to be used will be either PVC or high density polyethylene.

6-in diameter	18,300 m
8-in diameter	400 m
10-in diameter	870 m
12-in diameter	580 m

Lift stations

Due to the flat terrain two lift stations were required, one in the eastern and other in the western sectors of town. Each lift station is preceded by a communitor with a by-pass containing hand-cleaned bar screens. The wet wells were designed for 10-minute detention at average flow. Submersible pumps driven by electric motors will be used. The lift stations will be underground and will be installed as shown on Plan IV-23. Motor control center and liquid level control will be provided. A stand-by power plant with capacity to supply both lift stations will be installed.

Sewage outfall

As shown on Plan IV-30, the sewage will be pumped from the two wet wells to a 12-inch force main that ends in a manhole at the river bank from where it will flow by gravity to the outfall. The 12" Ø outfall line of PVC or high density polyethylene will advance 50 m into the river discharging through 10 diffusers set 3.0 m apart in the last 30 m of pipe.

Treatment

No treatment will be provided to the sewage prior to the discharge into the river for the following reasons:

a. The salinity of the Nickerie River at the point of discharge is such as to preclude the use of the water for irrigation or domestic purposes.

b. The muddy nature of the banks and bottom of the river makes it unattractive for recreational purposes.

c. There are no fishing or shellfish banks downstream from the outfall.

d. There is an average dilution of the sewage in the order of magnitude of 1:50,000.

e. The only beneficial use of the river at this point is navigation, which is not affected by sewage at this rate of dilution.

Cost Estimate

The estimated cost of the project is Sf.875,000 distributed as follows:

Collection system	Sf. 666,500	including house services
Lift stations	85,000	
Force main and outfall	20,000	
Contingencies, etc.	103,500	
	Sf. 875,000	

ALBINA SYSTEM

General

The drawings for the Albina proposed sanitary sewerage system are included as Plans IV-35 to -42. Due to the fact that the progress of the city is based on the development of the river side as a resort area using the beaches for recreation, only a very high quality effluent may be discharged into the river. For this reason treatment by stabilization ponds was included in the system. The city is served by a water supply system owned and operated by the Surinam Water Company. At present, the sewage is disposed of by means of septic tanks discharging either into infiltration fields or drainage ditches.

Location, Boundaries, and Description of the Area

Albina, the capital city of the District of Marowijne, is located on the left bank of the Marowijne River about 30 km from its mouth in the Atlantic Ocean. The area is very flat; the city is built on slightly higher ground than the adjacent areas that stay flooded during the rainy season. The present design covers the area served by the water supply system. Extensions will be possible in the future to serve areas of residential and industrial expansion outside the present city limits.

Population Served

The design population was estimated on the basis of the saturation of the area within the city limits. A population of 4,300 was obtained by this method and corresponds to the projection of 1988 of the present population of 3,000 at the growth rate of the last 10 years.

The development of a resort area on the riverside downstream from Albina is under study in the Planning Bureau of the Ministry of Public Works. This area is called Bamboo City and in the future will take care of the overflow population from Albina after urban saturation is reached in that town. Bamboo City will have its own independent sewerage system.

Design Analyses

Collection system

Based on data furnished by the Surinam Water Company on water consumption in Albina, the average design flow was estimated at 150 liters per capita per day; this includes domestic, commercial, and industrial contributions. A peaking factor of 2.0 was adopted for the collection system design. From this will result an average daily flow of 675 m^3, or 7.8 l/s and a peak flow of 15.6 l/s.

Design criteria for sewerage design were discussed in an earlier section.

In the design of the collection system an initial depth of 1.0 m was adopted to give adequate protection to the pipes.

Following are the pipe sizes and lengths included in the collection system and force main. Pipe to be used will be either PVC or high density polyethylene:

6-in diameter	8,500 m
8-in diameter	1,100 m

Lift station

Due to the flat terrain a lift station was required. The wet well was designed for 10 minutes' detention at average flow. Submersible pumps driven by electric motors will be used. The lift station will be underground and will be installed as shown on Plan IV-42. Motor control center and liquid level control will be provided. To ensure the reliability of operation a standby power plant will be installed.

Sewage treatment

A stabilization pond system was adopted as the means of sewage treatment. As shown in Plan IV-39, four ponds will be built—two anaerobic, one facultative, and one maturation.

For reasons of improving the operating conditions of the ponds they were designed for the present population instead of the future population, but the design has a built-in expansion capacity of 50%, bringing it up to the future needs with little construction work.

The treated effluent will be discharged into a ditch leading to the Anjoemara Creek, from which it goes to the Marowijne River.

Cost Estimate for Albina System

The estimated cost of the project is Sf.294,000, distributed as follows:

Collection system	Sf. 159,000
Lift station	55,000
Force main	19,000
Stabilization ponds	8,000
Contingencies, etc.	53,000
Total	Sf. 294,000

ANNEX 1

SEWER USE ORDINANCE

The responsible Government agency should adopt a sewer use ordinance which, with supporting regulations, will control the use of the public sewerage system and the private disposal of sewage and other liquid within its area of jurisdiction.

Article I - Definitions

Unless the context specifically indicates otherwise, the meaning of terms used in this ordinance shall be as follows:

Sec. 1. BOD (denoting Biochemical Oxygen Demand) shall mean the quantity of oxygen utilized in the biochemical oxidation of organic matter under standard laboratory procedure five days at 20°C, expressed in milligrams per liter.

Sec. 2. Building drain shall mean that part of the lowest horizontal piping of a drainage system which receives the discharge from soil, waste and/or other drainage pipes inside the walls of the building and conveys it to the building sewer, beginning one and half (1.5) meters outside the inner face of the building wall.

Sec. 3. Building sewer shall mean the extension from the building drain to the public sewer or other place of disposal.

Sec. 4. Combined sewer shall mean a sewer receiving both surface run-off and sewage.

Sec. 5. Garbage shall mean solid wastes from the domestic and commercial preparation, cooking, and dispensing of food, and from the handling, storage, and sale of products.

Sec. 6. Industrial wastes shall mean the liquid wastes from industrial manufacturing processes, trade, or business, as distinct from sanitary sewage.

Sec. 7. Natural outlet shall mean any outlet into a watercourse, pond, ditch, lake, or other body of surface or ground water.

Sec. 8. Person shall mean any individual, firm, company, associate, society, corporation, or group.

Sec. 9. Public sewer shall mean a sewer in which all owners of adjoining properties have equal rights and which is controlled by public authority.

Sec. 10. Sanitary sewer shall mean a sewer which carries sewage and to which storm, surface, and ground waters are not intentionally admitted.

Sec. 11. Sewage shall mean a combination of the water-carried wastes from residences, business buildings, institutions, and industrial establishments together with such ground, surface, and storm water as may be present.

Sec. 12. Sewage treatment plant shall mean any arrangement of devices and structures used for treating sewage.

Sec. 13. Sewage works shall mean all facilities for collecting, pumping, treating, and disposing of sewage.

Sec. 14. Sewer shall mean a pipe or conduit for carrying sewage.

Sec. 15. Shall is mandatory; may is permissive.

Sec. 16. Slug shall mean any discharge of water, sewage, or industrial waste which in concentration of any given constituent or in quantity of flow exceeds for any period of duration longer than fifteen (15) minutes more than five (5) times the average twenty-four (24) hour concentration or flow during normal operation.

Sec. 17. Storm drain (sometimes called storm sewer) shall mean a sewer that carries storm and surface water and drainage, but excludes sewage and industrial wastes other than unpolluted cooling water.

Sec. 18. Superintendent shall mean the Superintendent of Sewage Works or his authorized deputy, agent, or representative.

Sec. 19. Suspended solids shall mean solids that either float on the surface of, or are in suspension in, water, sewage, or other liquid and which are removable by laboratory filtering.

Sec. 20. Watercourse shall mean a channel in which a flow of water occurs, either continuously or intermittently.

Article II - Use of Public Sewers Required

Sec. 1. It shall be unlawful for any person to place, deposit, or permit to be deposited in any unsanitary manner on public or private property, any human or animal excrement, garbage, or other objectionable waste.

Sec. 2. It shall be unlawful to discharge to any natural outlet any sewage or other polluted waters, except where suitable treatment has been provided in accordance with subsequent provisions of this ordinance.

Sec. 3. Except as hereinafter provided, it shall be unlawful to construct or maintain any privy, privy vault, septic tank, cesspool, or other facility intended or used for the disposal of sewage.

Sec. 4. The owner of all houses, buildings, or properties used for human occupancy, employment, recreation, or other purposes, abutting on any street or right-of-way in which there is now located or may in the future be located a public sanitary or combined sewer, is hereby required, at his expense, to install suitable toilet facilities therein, and to connect such facilities directly with the proper public sewer in accordance with the provisions of this ordinance, within ... days after date of official notice to do so, provided that said public sewer is within thirty (30) meters of the property line.

<u>Article III - Private Sewage Disposal</u>

Sec. 1. Where a public sanitary or combined sewer is not available under the provisions of Article II, Section 4, the building sewer shall be connected to a private sewage disposal system complying with the provisions of this article.

Sec. 2. Before commencement of construction of a private disposal system the owner shall first obtain a written permit signed by the Superintendent. The application for such a permit shall be made on a form furnished by the responsible Government agency which the applicant shall supplement by any plans, specifications, and other information as are deemed necessary by the Superintendent. A permit and inspection fee of Sf. .... shall be paid to the agency at the time the application is filed.

Sec. 3. A permit for a private sewage disposal system shall not become effective until the installation is completed to the satisfaction of the Superintendent. He shall be allowed to inspect the work at any stage of construction and, in any event, the applicant for the permit shall notify the Superintendent when the work is ready for final inspection, and before any underground portions are covered. The inspection shall be made within ... hours of the receipt of the notice by the Superintendent.

Sec. 4. The type, capacities, location, and layout of a private sewage disposal system shall comply with all recommendations of the responsible public agency. No permit shall be issued for any private sewage disposal system employing subsurface soil absorption facilities where the area of the lot is less than ... square meters. No septic tank or cesspool shall be permitted to discharge to any natural outlet.

Sec. 5. At such a time as a public sewer becomes available to a property serviced by a private sewage disposal system as provided in Article III, Sec. 4, a direct connection shall be made to the public sewer in compliance with this ordinance, and any septic tanks, cesspools, and similar private sewage disposal facilities shall be abandoned and filled with suitable material.

Sec. 6. The owner shall operate and maintain the private sewage disposal facilities in a sanitary manner at all times, at no expense to the agency.

Sec. 7. No statement contained in this article shall be constructed to interfere with any additional requirements that may be imposed by the Health Officer.

Sec. 8. When a public sewer becomes available, the building sewer shall be connected to said sewer within ... days and the private sewage disposal system shall be cleaned of sludge and filled with clean gravel or dirt.

Article IV - Building Sewers and Connections

Sec. 1. No unauthorized person shall uncover, make any connections with or opening into, use, alter, or disturb any public sewer or appurtenance thereof without first obtaining a written permit from the Superintendent.

Sec. 2. There shall be two (2) classes of building sewers permits: a) for residential and commercial service, and b) for service to establishments producing industrial wastes. In either case, the owner, or his agent, shall make application on a special form furnished by the agency. The permit application shall be supplemented by any plans, specifications, or other information considered pertinent in the judgement of the Superintendent. A permit and inspection fee of Sf. .... for an industrial building sewer permit shall be paid to the agency at the time the application is filed.

Sec. 3. All costs and expense incident to the installation and connection of building sewer shall be borne by the owner. The owner shall indemnify the city from any loss or damage that may directly or indirectly be occasioned by the installation of the building sewer.

Sec. 4. A separate and independent building sewer shall be provided for every building; except where one building stands at the rear of another on an interior lot and no private sewer is available or can be constructed to the rear building through an adjoining alley, court, yard, or driveway, the building sewer from the front building may be extended to the rear building and the whole considered as one building sewer.

Sec. 5. Old building sewers may be used in connection with new buildings only when they are found, on examination and test by the Superintendent, to meet all requirements of this ordinance.

Sec. 6. The size, slope, alignment, materials of construction of a building sewer, and the methods to be used in excavating, placing the pipe, jointing, testing, and backfilling the trench shall all conform to the requirements of the building and plumbing code or other applicable rules and regulations of the agency.

Sec. 7. Whenever possible, the building sewer shall be brought to the building at an elevation below the basement floor. In all buildings in which any building drain is too low to permit gravity flow to the public sewer, sanitary sewage carried by such building drain shall be lifted by an approved means and discharged to the building sewer.

Sec. 8. No person shall make connection of roof downspouts, exterior foundation drains, or other sources of surface run-off or ground water to a building sewer or building drain which in turn is connected directly or indirectly to a public sanitary sewer.

Sec. 9. The connection of the building sewer into the public sewer shall conform to the requirements of the building and plumbing code or other applicable rules and regulations of the agency. All such connections shall be made gastight and watertight. Any deviation from the prescribed procedures and materials must be approved by the Superintendent.

Sec. 10. The applicant for the building sewer permit shall notify the Superintendent when the building sewer is ready for inspection and connection to the public sewer. The connection shall be made under the supervision of the Superintendent or his representative.

Sec. 11. All excavations for building sewer installation shall be adequately guarded with barricades and lights so as to protect the public from hazard. Streets, sidewalks, parkways, and other public properties disturbed in the course of the work shall be restored in a manner satisfactory to the agency.

Article V - Use of Public Sewers

Sec. 1. No person shall discharge or cause to be discharged any storm water, surface water, ground water, roof run-off, subsurface drainage, uncontaminated cooling water, or unpolluted industrial process water to any sanitary sewer.

Sec. 2. Storm water and all other unpolluted drainage shall be discharged to such sewers as are specifically designated storm sewers or combined sewers, or to a natural outlet approved by the Superintendent. Industrial cooling water or unpolluted process water may be discharged, on approval of the Superintendent, to a storm sewer, combined sewer, or natural outlet.

Sec. 3. No person shall discharge or cause to be discharged any of the following described waters or wastes to any public sewers:

a. Any gasoline, benzene, naphtha, fuel oil, or other flammable or explosive liquid, solid, or gas.

b. Any waters or wastes containing toxic or poisonous solids, liquids, or gases in sufficient quantity, either singly or by interaction with other wastes, to injure or interfere with any sewage treatment process, to constitute a hazard to humans or animals, create a public nuisance, or create any hazard in the receiving waters of the sewage treatment plant, including but not limited to cyanides in excess of two (2) milligrams per liter as CN in the wastes discharged to the public sewer.

c. Any water or wastes having a pH lower than 5.5 or having any other corrosive property capable of causing damage or hazard to structures, equipment, and personnel of the sewage works.

d. Solid or viscous substances in quantities or of such size capable of causing obstruction to the flow of sewers, or other interference with the proper operation of the sewage works such as, but not limited to, ashes, cinders, sand, mud, straw, shavings, metal, glass, rags, feathers, tar, plastics, wood, garbage, whole blood, paunch manure, hair and entrails, paper dishes, cups, milk containers, etc.

Sec. 4. No person shall discharge or cause to be discharged the following described substances, materials, waters, or wastes if it appears likely in the opinion of the Superintendent that such wastes can harm either the sewers, sewage treatment process, or pumps and equipment, have an adverse effect on the receiving stream, or otherwise endanger life, limb, public property, or constitute a nuisance. In forming his opinion as to the acceptability of these wastes, the Superintendent will give consideration to such factors as the quantities of subject wastes in relation to flows and velocities in the sewers, material of construction of the sewers, nature of the sewage process treatment, capacity of the sewage treatment plant, and other pertinent factors. The substances prohibited are:

a. Any liquid or vapor having a temperature higher than sixty-five (65)°C.

b. Any water or waste containing fats, wax, grease, or oils, whether emulsified or not in excess of one hundred (100) mg/1, or containing substances which may solidify or become viscous at temperatures between zero (0) and sixty-five (65)^{o}C.

c. Any garbage.

d. Any waters or wastes containing strong acid iron pickling wastes or concentrated plating solutions, whether neutralized or not.

e. Any waters having a pH higher than 9.5.

f. Any waters or wastes containing iron, chromium, copper, zinc, and similar objectionable or toxic substances, or wastes exerting an excessive chlorine requirement, to such degree that any such material received in the composite sewage at the sewage treatment plant exceeds the limits established by the Superintendent for such materials.

g. Materials that exert or cause:

1. Unusual concentrations of inert suspended solids (such as, but not limited to, Fullers' earth, lime slurries, and lime residues) or of dissolved solids (such as, but not limited to, sodium chloride and sodium sulfate).

2. Excessive discoloration (such as, but not limited to, dye wastes and vegetable tanning solutions).

3. Unusual volume of flow or concentration of wastes constituting "slugs" as defined herein.

Sec. 5. If any wastes are discharged, or are proposed to be discharged to the public sewers, which waters contain the substances or possess the characteristics enumerated in Sec. 4 of this Article, and which, in the judgement of the Superintendent, may have a deleterious effect upon the sewage works, equipment, or receiving waters or which otherwise create hazard to life or constitute a public nuisance, the Superintendent may:

a. Reject the wastes.

b. Require pretreatment to an acceptable condition for discharge to the public sewers.

c. Require control over the quantities and rates of discharge.

If the Superintendent permits the pretreatment or equalization of waste flows, the design and installation of the plants and equipment shall be subject to the review and approval of the Superintendent, and subject to the requirements of all applicable codes, ordinances, and laws.

Sec. 6. Grease, oil, and sand interceptors shall be provided when, in the opinion of the Superintendent, they are necessary for the proper handling of liquid wastes containing grease in excessive amounts, or any flammable wastes, sand, or other harmful ingredients; except that such interceptors shall not be required for private living quarters or dwelling units. All interceptors shall be of a type and capacity approved by the Superintendent, and shall be located as to be readily and easily accessible for cleaning and inspection.

Sec. 7. Where preliminary treatment or flow-equalizing facilities are provided for any waters or wastes, they shall be maintained continuously in satisfactory and effective operation by the owner at his expense.

Sec. 8. When required by the Superintendent, the owner of any property serviced by a building sewer carrying industrial wastes shall install a suitable control manhole together with such necessary meters and other appurtenances in the building sewer to facilitate observation, sampling, and measurement of the wastes. Such manhole when required shall be accessibly and safely located and shall be constructed in accordance with plans approved by the Superintendent. The manhole shall be installed by the owner at his expense, and shall be maintained by him so as to be safe and accessible all the time.

Sec. 9. All measurements, tests, and analyses of the characteristics of waters and wastes to which reference is made in this ordinance shall be determined in accordance with the latest edition of "Standard Methods for the Examination of Water and Waste Water," published by the American Public Health Association, and shall be determined at the control manhole provided, or upon suitable samples taken at said control manhole. In the event that no control manhole has been required, the control manhole shall be considered to be the nearest downstream manhole in the public sewer to the point at which the building sewer is connected. Sampling shall be carried out by customarily accepted methods to reflect the effect of constituents upon the sewage works, and to determine the existence of hazard to life, limb, and property. (The particular analyses involved will determine whether a twenty-four (24) hour composite of all outfalls of a premise is appropriate or whether a grab sample or samples should be taken. Normally, but not always, BOD and suspended solids analyses are obtained from 24-hour composite of all outfall, whereas pH's are determined from periodic grab samples.)

Article VI - Protection from Damage

Sec. 1. No unauthorized person shall maliciously, willfully, or negligently break, damage, destroy, uncover, deface, or tamper with any structure, appurtenance, or equipment which is a part of the sewage works. Any person violating this provision shall be subject to immediate arrest under charge of disorderly conduct.

Article VII - Power and Authority of Inspectors

Sec. 1. The Superintendent and other duly authorized employees of the agency bearing proper credentials and identification shall be permitted to enter all properties for the purposes of inspection, observation, measurement, sampling, and testing, in accordance with the provisions of this ordinance.

Article VIII - Penalties

Sec. 1. Any person found to be violating any provision of this ordinance except Article VI shall be served by the agency with written notice stating the nature of the violation and providing a reasonable time limit for the satisfactory correction thereof. The offender shall within the period of time stated in such notice, permanently cease all violations.

Sec. 2. Any person who shall continue any violation beyond the time limit provided for in Article VIII, Section 1, shall be guilty of a misdemeanor, and on conviction thereof shall be fined in the amount not exceeding Sf. .... for each violation. Each day in which any such violation shall continue shall be deemed a separate offense.

Sec. 3. Any person violating any of the provisions of this ordinance shall become liable to the agency for any expense, loss, or damage caused the agency by reason of such violation.

Article IX

Sec. 1. All by-laws or parts of by-laws in conflict herewith are hereby repealed.

Sec. 2. The invalidity of any section, clause, sentence, or provision of this ordinance shall not affect the validity of any other part of this ordinance which can be given effect without such invalid part or parts.

ANNEX 2

DESIGN BASIS - SURFACE WATER TREATMENT PLANT
(SANTIGRON)

TOTAL FLOW FOR MASTER PLAN STUDY

For flow design a constant figure of 100 l/cap/day has been used as an average during a 24-hour period.

Q for 1987 = 10,000 m^3/day (24-hour pumping rate)
Q for 1987 = 15,000 m^3/day (16-hour pumping rate) = 4,150 gpm
Q for 2001 = 17,000 m^3/day (24-hour pumping rate)
Q for 2001 = 25,500 m^3/day (16-hour pumping rate) = 7,050 gpm

a. Overload Factor: The plant has been designed with an overload factor of 25%.

		Rating			Overload		
	Year	Mgd	Gpm	Cfs	Mgd	Gpm	Cfs
Stage I	1987	4.0	4,150	6.2	5.0	6,200	7.7
Stage II	2001	6.8	7,050	10.5	8.5	8,810	12.6

b. Clarifier Rating: 1.0 gpm/ft^2

Detention time: 2 hrs = 120 min

c. Flash Mixer: Detention time: 1/2 min

d. Rapid Gravity Filters: Loading rate : 2.0 gpm/ft^2
Backwash rate: 20.0 gpm/ft^2

e. Sludge Drying Beds (if used): Loading rate: 2.0 mgd/acre

f. Backwash Rate Beds (if used): Loading rate: 3.0 mgd/acre

DESIGN SURFACE WATER TREATMENT PLANT

a. Clarifier: 1987 - 1 unit
2001 - 2 units

For year 1987 - 90-ft diameter circular tank
Capacity (including 1% additional capacity for sludge):

625,000 + 6,250 = 631,250 gallons

Depth (including 2-ft free board)

14.0 + 2.0 = 16.0 ft

Vertical velocity - $\pm$ 1"/min

For year 2001 - add 1 unit - 75-ft diameter circular tank
Capacity (including 1% additional capacity for sludge):

435,000 + 4,350 = 439,350 gallons

Depth (including 2-ft free board)

14.0 + 2.0 = 16 ft

Vertical velocity - $\pm$ 1"/min

b. Flash Mixer: The same tank will be used for the 1987 and 2001 designs. The flash mixer has been designed with four separate outflow structures, each with a removable weir plate at the intake to regulate the flow to the clarifier.

The tank has an approximate size of 10 x 10 ft and a depth of 6.5 ft.

The detention time for 1987 is $\pm$ 0.85 min, somewhat larger than the 0.5 min required by the basis of design. Detention time for 2001 is $\pm$ 0.50 min.

Until 1987, only two outlet structures are to be used. The two remaining are to be ones plugged until after year 1987. The tank has been designed with 18" free board above its operational water level.

c. Rapid Gravity Filters: Backwash for the filters will be provided by an elevated tank with a capacity of 50,000 gallons. After 1987 a second elevated tank with the same capacity has to be erected to serve the additional filters required by 2001. The 1987 capacity total is 5,200 gpm.

The filters are designed in such a manner that each filter is out of service for 15 min/day. For a 16-hour plant operation this amounts to

16 x 4 = 64 quarter hours of time, or 1/64 x 100% = 1.5% of a 16-hour run.

Five filters are planned, and, if washed every day, the total surface area has to be enlarged by $\frac{1.5}{5}$ 0.3%.

Area of each filter (including 0.3% additional total area):

$$\frac{2,600 + 8}{5} = \frac{2,608 \text{ ft}^2}{5} = 522 \text{ ft}^2 = 49 \text{ m}^2, \text{ use } 5 \times 10 \text{ m}$$

The backwash flow for each filter = 522 x 20.0 = 10,440 gpm

The elevated tank capacity is 50,000 gallons. Time to empty tank $= \frac{50,000}{10,440} = 4.8$ min, say 5.0 min.

The backwash volume required/day is 5 x 50,000 = 250,000 gallons.

Two main gutters are designed for each filter, with dimensions of 1.5 x 2.85 ft (including 0.25 ft of free board). The capacity of each is 5,220 gpm. Five side gutters are designed for each filter, with dimensions of 1.0 x 1.40 ft (including 0.25-ft free board). The capacity of each is 1,045 gpm.

Five additional filters are planned for the year 2001 requirement, necessitating provision of another 50,000-gallon elevated tank.

The additional capacity required is 8,810 - 5,200 gpm = 3,610 gpm.

The area of each filter (including 0.3% additional total area):

$$\frac{1,805 + 5.4}{5} = 362 \text{ ft}^2 = 34 \text{ m}^2, \text{ use } 5 \times 7 \text{ m}$$

The backwash flow for each filter = 362 x 20.0 = 7,240 gpm with the elevated tank capacity of 50,000 gallons, the time required to empty tank $= \frac{50,000}{7,240} = 6.9$ min, say 7.0 min.

The backwash volume required/day is 5 x 50,000 = 250,000 gallons.

Two main wash gutters are designed for each filter, with dimensions of 1.5 x 2.30 ft (including 0.25-ft free board). The capacity of each is 3,620 gpm. Four side gutters are designed for each filter, with dimensions of 1.0 x 1.55 ft (including 0.25-ft free board). Capacity each 905 gpm.

d. Chemicals: Assumptions were made of the chemical requirements in lb/day. Actual testing of the Saramacca River water was limited to that performed at Uitkijk and only for pH, Cl, and Fe. Project staff found that during the months of May and June 1970, the average pH was approximately 6.0, Cl was approximately 17 ppm, and Fe was approximately 0.15 ppm. An extensive laboratory testing program will have to be initiated by the Government at Santigron should conditions develop which require the selection of this surface water treatment plant to supply the rural areas in the manner defined in this master plan study.

The following approximate chemical requirements were utilized in order to arrive at operational cost estimates. Exact requirements would, of course, be available following extensive analysis of the raw water.

Chemical	Max. Dosage ppm	1987 Q = 5.0 mgd	2001 Q = 8.5 mgd
Chlorine			
pre	5.0	5x5x8.33=210 lb	5x8.5x8.33=355 lb
post	1.5	1.5x5x8.33=62.5 lb	1.5x8.5x8.33=107 lb
Alum	35	35x5x8.33=1,460 lb	35x8.5x8.33=2,480 lb
Lime			
Flocculation	15	15x5x8.33=625 lb	15x8.5x8.33=1,070 lb
Corrosion control	20	20x5x8.33=833 lb	20x8.5x8.33=1,420 lb

1 ppm = 8.33 lb/million gallons

(1) For year 1987 the alum [Al_2 (SO_4) 3.14 H_2O] requirement is as follows:

The required 1,460 pounds is to be applied in 24 hours since project staff assumed only 16 hours operation (two work shifts):

$$\frac{24}{16} \times 1{,}460 \text{ lb} = 2{,}200 \text{ lb to be applied in 16 hours}$$

Liquid alum [Al_2 (SO_4) 3.14 H_2O] with a concentration of 35% dry aluminum sulfate contains 3.85 lb.

At $2{,}200 \text{ lb/day} \times 30 \times \frac{1 \text{ gal}}{3.85 \text{ lb}} = 17{,}200$ gallons dry aluminum sulfate required/month.

Since one gallon of dry aluminum sulfate weighs 11.3 lb, the total required per month is 17,200 x 11.3 lb = 194,000 lb.

Alum is available in 50-kg bags. Therefore, the total required per month is approximately 1,760 bags of 50 kg each.

For year 2001 the alum requirement is as follows:

For a 16-hour-per-day operation the requirement is $\frac{24}{16}$ x 2,480 lb = 3,700 lb/day.

Dry aluminum sulfate required per month:

$$3,700 \text{ lb/day} \times 30 \times \frac{1 \text{ gal}}{3.85 \text{ lb}} = 28,800 \text{ gallons}$$

Total additional volume required: 28.800 - 17,200 = 11,600 gallons per month.

$$11,600 \times 11.3 \text{ lb} = 131,000 \text{ lb/month}$$

Additional number of 50-kg bags required - 1,200/month

(2) For year 1987 the chlorine requirement is as follows:

Chlorination (use 1-ton containers)

The requirement for prechlorination is 210 lb and for postchlorination is 62.5 lb per 24 hours since project staff assumed only 16 hours operation (two work shifts) for prechlorination:

Prechlorination:	$\frac{24}{16}$ x 210 lb =	315.0	lb
Postchlorination:		62.5	lb
	Total	377.5	lb/day

If six one-ton containers are used, the supply will last $\frac{13,200 \text{ lb}}{377.5 \text{ lb/day}}$ = 35 days. Approximate one-month supply.

For year 2001, the chlorine requirement is as follows:

- Same requirement on operational time as 1987.

$$\text{Prechlorination: } \frac{24}{16} \times 355 \text{ lb} = 530 \text{ lb}$$

Postchlorination:		107 lb
	Total	637 lb/day

If ten one-ton containers are used, supply will last $\frac{22,000 \text{ lb}}{637 \text{ lb/day}}$ = 34.5 days. Approximate one month supply.

(3) For year 1987 the lime requirement is as follows:

Lime (quick lime)

The requirement for flocculation is 625 lb and for corrosion control 833 lb per 24 hours since project staff assumed only 16 hours operation (two work shifts) for flocculation.

$$\text{Flocculation: } \frac{24}{16} \times 625 \text{ lb} = 940 \text{ lb}$$

Corrossion control:		833 lb
	Total	1,773 lb/day

The weight of dry lime is 40 lb/ft^3 with a density of 96%

Quick lime density is approximate 73%

Total volume required is $\frac{1,773}{40} \times \frac{.73}{.96} = 34.0 \text{ ft}^3\text{/day}$

Total storage volume for one month: $30 \times 34 = 1,020 \text{ ft}^3$

For year 2001 the lime requirement is as follows:

Same requirements on operational time as 1987

$$\text{Flocculation: } \frac{24}{16} \times 1,070 \text{ lb} = 1,600 \text{ lb}$$

Corrosion control:		1,420 lb
	Total	3,020 lb/day

Total additional volume required:

$$\frac{(3,020-1,773)}{40} \times \frac{.73}{.96} = 23.7 \text{ ft}^3\text{/day}$$

Total additional volume for one month: $30 \times 23.7 = 711 \text{ ft}^3$

e. <u>Ground and Elevated Storage</u>: In order to establish the required storage at the filter plant, the entire master plan area had to be examined. Following are the required storage volumes for the major areas to be served:

Area	1987 Supply m^3/day	1987 Ground Storage m^3	1987 Elevated Storage m^3	2001 Ground Storage m^3	2001 Elevated Storage m^3
Pad van Wanica	2,500	1,250	125	2,100	210
Commewijne	5,000	2,500	250	4,200	420
Future developments	2,500	1,250	125	2,100	210
Total	10,000	5,000	500	8,400	840
		500		840	
		5,500		9,240	

Total for 1987: 5,500
Total for 2001: 9,240

Storage for 1987

Since no reservoirs were planned at the sites to be developed in the future due to their scattered locations, storage was planned at the filter plant. In the Commewijne area only a small ground storage reservoir was planned for 1987 at Meerzorg, with a capacity including the elevated tank of approximately 845 m^3.

Total storage volume has been planned at the filter plant of 60% x 5,500 m^3 = 3,330 m^3. Three separate reservoirs of 12.0 x 18.5 x 5.0 m should be able to handle this storage requirement. No elevated tanks were planned. Booster pumps were chosen to maintain the pressure in the main feeder serving the master plan areas.

Storage for 2001

After 1987 the pressure in the main feeder would drop considerably because of the higher water demands. A booster station has been planned at the existing Kwatta-Leidingen treatment plant to increase the pressure in order to provide adequate flows in the Commewijne area. By this time enough storage capacity will have been built in Commewijne to provide an adequate factor of safety in the water supply in case of power or equipment failures. Therefore, no additional storage tanks are to be built at the filter plant site.

It is planned that the Pad van Wanica area will have its own storage reservoir with a year 2001 expansion possibility. Project staff gave special emphasis to the need for backup in case of failure at the booster-plant location of the surface water treatment plant.

ANNEX 3

DESIGN BASIS - GROUND WATER TREATMENT PLANT
(KWATTA-LEIDINGEN)

SOURCE

Ground water has been used as a source for this treatment plant. Elevation of ground water before pumping is approximately 2.0 meters below the ground surface. Drawdown at the maximum allowable pumping rate for this well is approximately 500 meters.

Chemical quality was found as follows:

pH	Cl-	Fe +++	SO_4 --	Residue
7	162 ppm	3.0	179	770

FLOW DESIGN

The following basis has been used for flow design:

Period	Average Daily Consumption	Population Growth (%)
1973 - 1980	80 l/c/d	4
1980 - 1987	100 l/c/d	4
1987 - 2002	125 l/c/d	4

SCHEDULE FOR INSTALLATION

Stage 1 to be installed in 1973 (present)
Stage 2 to be installed in 1980
Stage 3 to be installed in 1987

DESIGN POPULATION

Stage 1 is design for population in 1980
Stage 2 is design for population in 1987
Stage 3 is design for population in 2002

	Design Year	Total Population	% Connected	Population Connected
Stage 1 (present)	1980	17,000	60	10,200
Stage 2 (1980)	1987	25,000*	95	23,800
Stage 3 (1987)	2002	45,000	100	45,000

*Includes 2,500 for possible Jarikaba extension

Water Consumption	Year	Average Daily Consumption	Rating 18 Hours of Pumping			
			m 3/hr	Mgd	gpm	cfs
Stage 1	1973	820 m^3	45.5	.29	200	.45
Stage 2	1980	2,380 m^3	132	.83	575	1.29
Stage 3	1987	5,600 m^3	310	1.95	1,350	3.02

DESIGN OF WELL PUMP

Stage 1

Q = 200 gpm

T.D.H = 55 ft. n = 0.60

$$\text{H.P.} \quad \frac{200 \times 55}{3{,}970 \times 0.60} = 4.6 \text{ (use 7.5 H.P.)}$$

This pump is capable of delivering 200 gpm with H = 90 ft. (± 35 ft over pressure).

Stage 2

Q = 575 gpm

T.D.H. = 70 ft. n = 0.74

$$\text{H.P.} \quad \frac{575 \times 70}{3{,}970 \times 0.74} = 13.5 \text{ (use 15 H.P.)}$$

TREATMENT PLANT PROCESS

Aeration is used to reduce the iron concentration from the unacceptable level of approximately 3.0 ppm to a level of approximately 0.3 ppm. It also removes the dissolved gases such as Co_2 and H_2 S which are present in small quantities. With the typical aeration approach used on this filter plant the taste and odor of the ground water will be improved considerably.

Spray aeration of the water has been applied on this project using spray nozzles with a pressure at the nozzles of approximately 8 psi. The diameters of the nozzles are kept in the 1" range in order to prevent clogging. The nozzles are located some 4 feet above the maximum water level in the aeration chambers. The chambers have an operating depth of approximately 6 feet. The aeration structure has been fitted with louvers to allow air circulation and containment of the water.

After oxidation of iron and removal of the gases, the effluent is discharged into sand filters (rapid gravity sand filters). These sand filters are essential in order to remove the iron particles remaining in suspension after the aeration process. The filter bed for this treatment plant will have sand with a uniformity coefficient of approximately 1.6 and a depth of 3 feet. Total gravel depth has been kept at 1 1/2 feet. The filters have been designed with an underdrain system consisting of main drain with laterals for each filter compartment. All underdrains are perforated at the bottom, ensuring uniform withdrawal of the water and uniform distribution of the wash water without excessive loss of head.

It was found that after sand filtration the pH dropped somewhat in most treatment plants in the country. It is therefore a general practice to "polish" the filtered effluent by discharging the sand-filtered water in shell filters. This brings the pH up again and improves the overall quality of the water, including taste. The size of the shell filters is kept the same as designed for the sand filters. Shells for this purpose are readily available in the country and therefore are relatively cheap for this purpose. Backwash capabilities are the same for both sand and shell filters. It is the intention to use only one sand filter first and make complete chemical analyses of its effluent. After this is known, decisions will be made to use a second sand filter or a shell filter instead.

There is enough flexibility in the plant to make all necessary adjustments as far as filtration is concerned.

All filtered water will be stored in ground storage tanks located adjacent to the filters. From here the water will be pumped into an elevated storage tank which provides the distribution system with reliable pressures and flows.

Water levels in the plant are electronically controlled by electrodes which are also connected to the source of electrical supply. The electrodes in the filter and aeration units control the well pumps. The electrodes in the elevated tank control the ground storage pumps.

The electrical system has been designed in such a manner that it can be connected to a portable generator in case of electrical line system failure.

AERATION UNITS

The aeration units have been designed for a flow based on an 18-hour pumping rate in stage 2 (1987).

Use 24 nozzles   No. 15 B nozzles
Capacity each nozzle = 24 gpm
Total   Q = 575 gpm

After 1987 an additional capacity of 775 gpm will be required.

Space for extension for this future aeration room is provided in front of existing aeration room.

SAND AND SHELL FILTERS

The rapid sand filters have been designed for a flow based on an 18-hour pumping rate in stage 2 (1987).

Loading rate   2 gpm/sq. ft.
Backwash rate   22.5 gpm/sq. ft.
Area required $\frac{575}{2} = 290$ sq. ft.

Two filters were recommended (one sand filter and one shell filter)

Area each filter   320 sq. ft.

After 1987 an additional area of 355 sq. ft. will be required. Space for extension is allowed in the design.

Future area will be 640 sq. ft.

Loading rate $\frac{1350}{640} = 2.14$ gpm/sq. ft.

BACKWASHING

Backwashing should be done in offpeak hours using water from the elevated storage tank.

Washing time approximately 5 min. (washing each filter at a different time).

Capacity of elevated storage tank is 40,000 gallons.

Total volume of water required for backwashing is 36,000 gallons.

In offpeak hours the total capacity of the elevated storage tank can be used without refilling the tank.

Wash gutters were designed using the empirical formula: $Q=1.91\ BZ^{1.5}$

CLEAR WATER STORAGE

a) Ground storage tank

The capacity of the ground storage tank has been based on 50% of the average daily consumption in 1987.

50% of 2,380 m^3 = 119 m^3 (use 40,000 gallons = 150 m^3)

For year 2002:

5% of 5,600 = 280 m^3 (use additional 150 m^3)

Space for extension is provided behind existing elevated storage.

CLEAR WATER PUMPS

Turbine pumps were recommended for pumping from clear water storage to the elevated storage.

Capacity based on an 18-hour pumping rate:

Stage 1

Q = 200 gpm

T.D.H. = 160 ft. n = 0.68

$$H.P. = \frac{200 \times 160}{3,970 \times 0.68} = 11.8 \text{ (use 12 H.P.)}$$

Stage 2

Q = 575 gpm

T.D.H. = 160 ft.

$$H.P. = \frac{575 \times 160}{3{,}970 \times 0.68} = 34 \text{ (use 35 H.P.)}$$

DISTRIBUTION SYSTEM

Analysis of the distribution system for pipe size determination was made on the basis of the Hardy Cross method.

Rough calculations were made using the following criteria:

1 Peak flow = 1,150 gpm*

2 No pipe size smaller than 3" was used.

3 Pressure at the most distant point is 20 psi. (at one junction 30 psi)

4 C = 120 (roughness coefficient for A.C. and PVC pipe)

Computer analysis was used to check preliminary calculations.

The computer program used was a node-balancing type of analysis, based on a modified Hardy Cross method.

PVC pipe was recommended from 3-in to 6-in diameter and A.C. pipe for 8-in to 10-in diameter.

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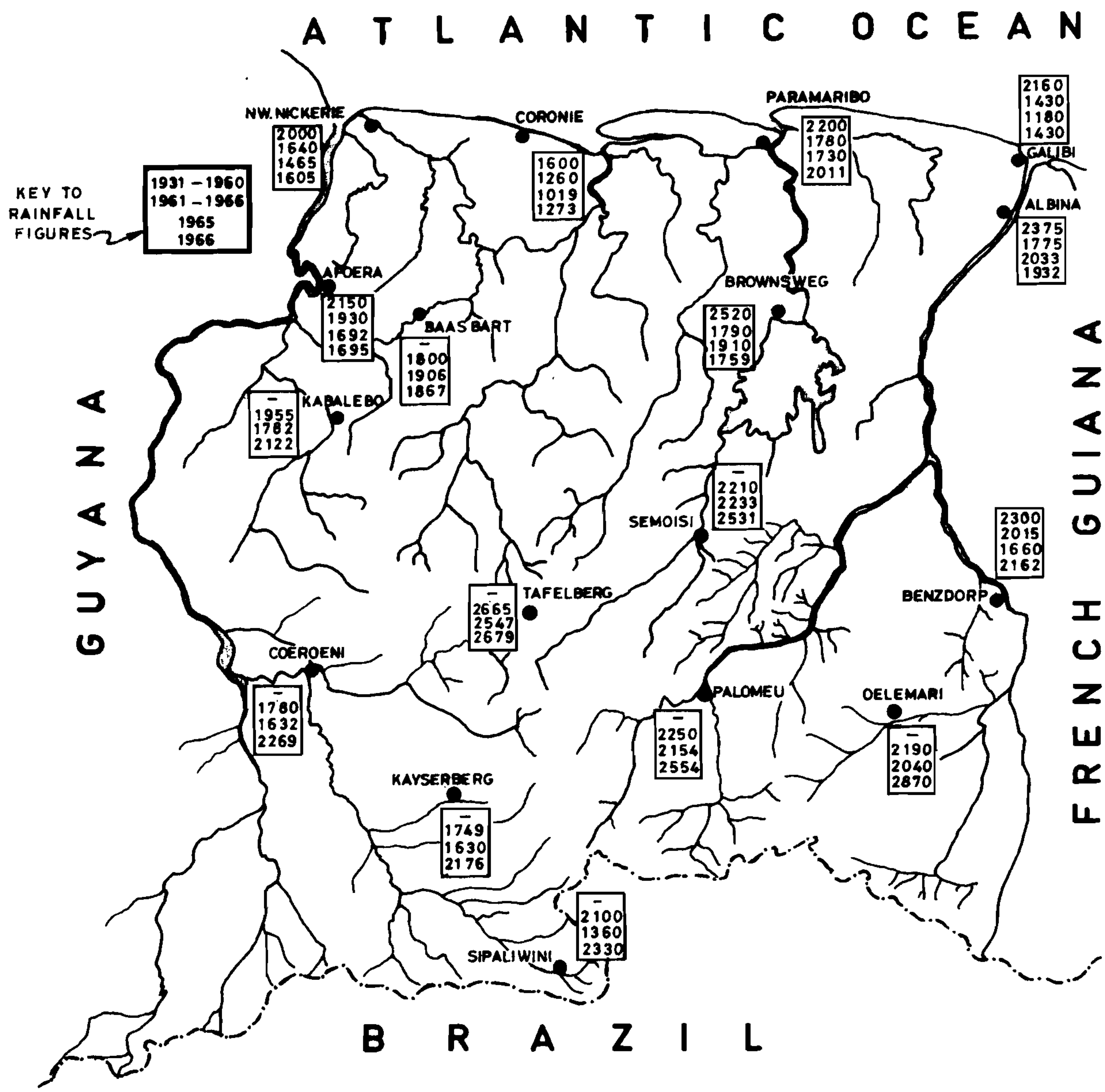
* Peak factor: 2 1/2

$$\text{Peak flow} : = \frac{2{,}300 \times 2\ 1/2}{24} = 250\ m^3/hr$$

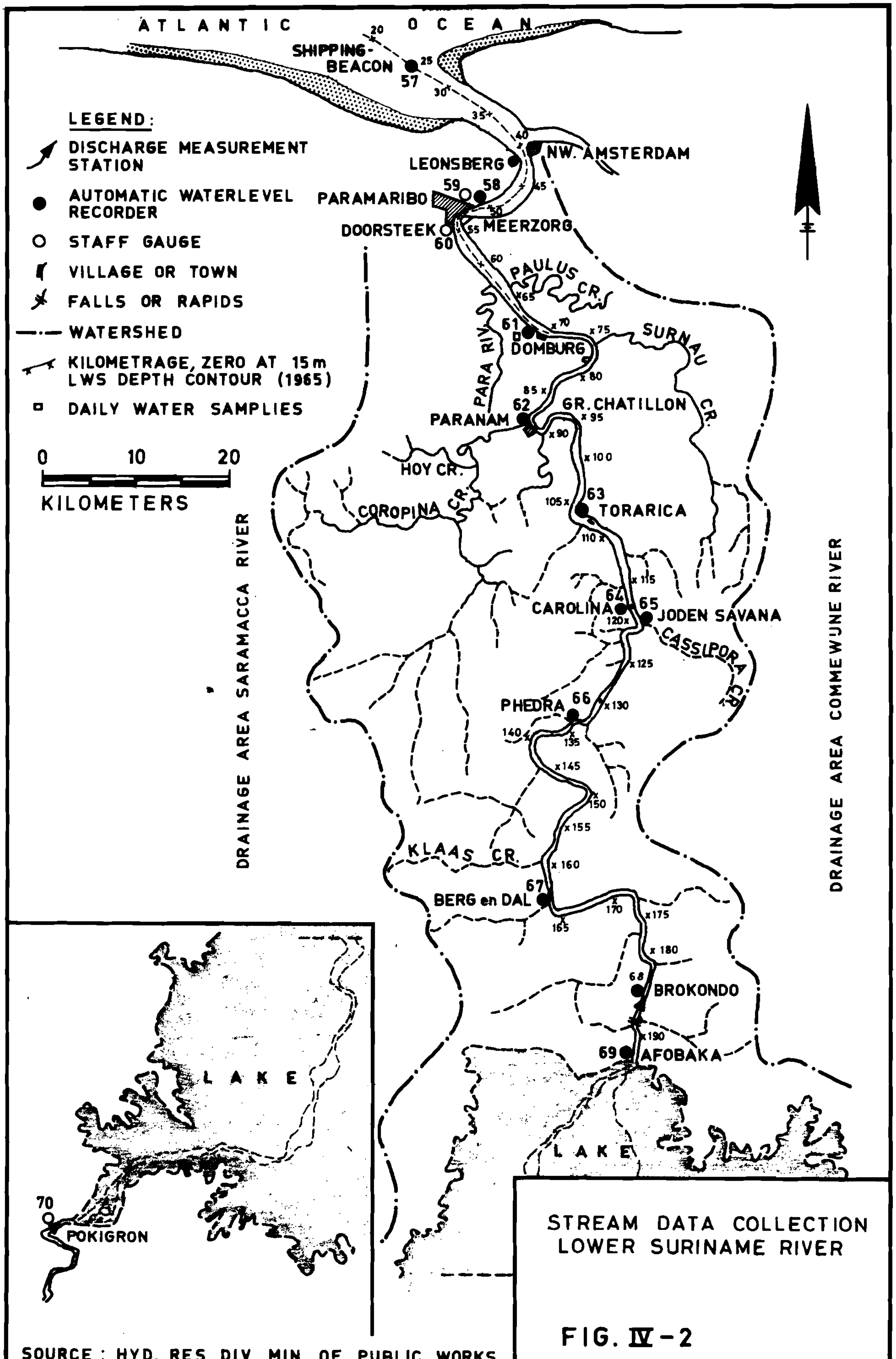
$$= 1{,}100 \text{ gpm} \qquad \text{(1,150 gpm was used)}$$



ANNUAL RAINFALL (mm) SURINAM
(PERIODS 1931- 1960, 1961 - 1966, 1965 AND 1966),
AND LOCATION OF THE MAIN DISCHARGE GAUGING STATIONS (●)

ANNUAL RAINFALL
SURINAM

FIG. IV-1



STREAM DATA COLLECTION
LOWER SURINAME RIVER

FIG. IV-2