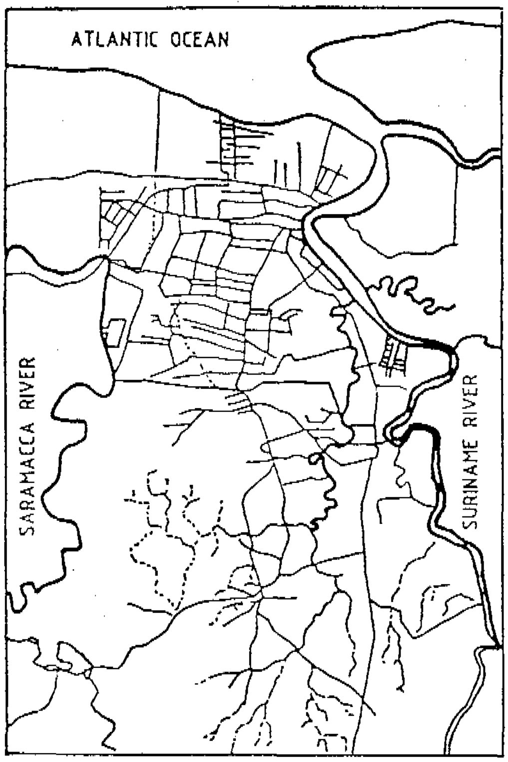
N.V. Surinaamsche Waterleiding Maatschappij

INTER-AMERICAN DEVELOPMENT BANK

Water Supply System for Paramaribo and its Metropolitan Area



FINAL REPORT, VOLUME 1: SUMMARY

November 1991

HARRIS
Frederic R. Harris, inc.,
New York, The Hague

IWACO
Consultants for Water & Environment
Rotterdam, The Netherlands

in association with SUNECON-Paramaribo

LIST OF VOLUMES

TABLE OF CONTENTS, VOLUME 1: SUMMARY

LIST OF TABLES

LIST OF FIGURES

1 DESCRIPTION OF THE PROJECT

1.1 INTRODUCTION

The extension and rehabilitation of the water supply system of Paramaribo has long been overdue. This was mainly caused by political and economic reasons.
By the end of the seventies, a crash programme (Livorno, realized early 1982) and a feasibility study were initiated in the framework of the cooperation between Suriname and The Netherlands. Due to political reasons, the implementation of the further extension works, proposed at that time, did not materialize.

In 1984 contact was made with the Inter-American Development Bank (IADB) for the implementation of an extension project. At the request of the IADB a new feasibility study was defined, which was realized by mid 1988. Following this feasibility study, an extensive detailed design phase was started by mid 1990, whereby the demand analysis and the source selection of the feasibility study had to be reconsidered drastically. During this detailed design phase, specific attention was also paid to the rehabilitation of existing production facilities.

The SWM has during the last years, within its financial and technical capabilities, constructed a number of small temporary production units, thereby reducing slightly the shortcomings in the production capacity of the system. These temporary production units are made of steel containers, which have an expected technical lifetime of approximately 5 years only and are constructed at considerable cost. After completion of the envisaged project these temporary production facilities will be taken out of use.
Furthermore, the old production facilities at Leysweg will be taken out of use as well, due to the quality of the raw water.

1.2 OBJECTIVES

The ultimate objective of the Paramaribo Water Supply Project is the immediate extension of the water supply system to meet the demand for potable water of the year 2000 in the first phase, and the forecast of the required extensions to meet the 2010 demand through a second phase.

The immediate objectives of the present Detailed Design Phase are:

1. A review and update of the technical, economic and financial data contained in the Final Report of the Feasibility Study, including Addendum, through complementary studies. Special emphasis will be given to the socio-economic aspects in relation to the demand forecasts and the hydrogeological mechanism of the Zanderij aquifer.

2. Final designs for the development of a new well-field in the Zanderij aquifer, a new treatment plant, and a new transmission pipeline as the main components in the expansion of the Water Supply System of Paramaribo. All the project components will be designed for their execution in two phases of ten years each and will be based on lease cost solutions. Works designed for the first phase will be presented as final designs and tender documents for international competitive bidding will be prepared. For the second phase, general engineering drawings of the proposed works will be prepared.

3. A mathematical model of the Zanderij Aquifer as a groundwater management tool.

4. Final designs for the rehabilitation of the existing well-fields, treatment plants and pumping stations.

5. Construction drawings for upgrading and rehabilitation of the existing water distribution network.

6. Preparing final designs for the distribution system of the developed areas located between the Zanderij International Airport and Lelydorp.

7. Advise the SWM on the Operation and Maintenance of the rehabilitated, upgraded and expanded water supply system, and prepare and implement a programme to control and reduce the percentage of unaccounted for water.

1.3 THE ENVISAGED PROJECT

The project as defined during the Detailed Design phase consists of the following main components:

1 New Well-field (WF)
2 New Water Treatment Plant (WTP)
3 Clear Water Transmission Main: van Hattemweg - WK-Plein (CWT 1)
4 Distribution Centre WK-Plein (DC)
5 Clear Water Transmission Main: WK-Plein - Blauwgrond (CWT 2)
6 Distribution System (DIS)
7 Miscellaneous Pumping Systems (MPS)
8 Rehabilitation Works Republiek (REHAB REP)
9 Rehabilitation Works WK-Plein (REHAB WK)
10 Rehabilitation Works Livorno (REHAB LIV)

In the following paragraphs a short description will be given of each of these main components.

1.3.1 NEW WELL-FIELD

The new well-field is projected along the van Hattemweg starting approximately 1,000 m from the Indira Ghandiweg, the main road between the international airport and the city.
The total required flow amounts to 2,000 m^3/hr, which will be provided by 20 wells, each designed to produce 100 m^3/hr. Two additional wells will be provided as stand-by. The wells are divided into two groups of 11 wells, each pumping the raw water into their own PVC header, which will convey the water to the new Water Treatment Plant.
The well-field has a total developed length of 8,100 m.

1.3.2 NEW WATER TREATMENT PLANT

The new Water Treatment Plant will be located along the van Hattemweg as well, at a distance of approximately 800 m from the Indira Ghandiweg. The treatment plant consists of the following process steps:

1. Aeration:
   The aeration will be realized by natural ventilation in order to avoid the use of vulnerable mechanical equipment and the risk of clogging.
2. Rapid sand filtration:
   The filtration unit consists of 8 open, concrete filter boxes. The filters are operated as downflow, constant rate filters, at a filtration rate of 5 m/hr. The filter medium is sand, 1-2 mm with a bed thickness of 1.5 m. Backwashing will be carried out with water (30 m^3/hr) and air.
3. Shell filtration:
   There are 4 open shell filters, of the same configuration as the sand filters. The filtration rate is 10 m/hr and with a minimum contact time of 5 minutes the resulting minimum bed thickness is 0.85 m. The maximum bed thickness amounts to 2 m, which means that the shell filters will have to be refilled every 2 to 3 months. The shell filters can be backwashed with water at a rate of 30 m^3/hr.
4. Disinfection:
   Disinfection will be carried out using sodium hypochlorite, which is locally produced and readily available.

After the conditioning step in the shell filters, the water will be conveyed to the Clear Water Reservoir CWR, which has a total volume of 6,100 m^3. This CWR is divided into 3 compartments of:

* 1,500 m^3 for not-disinfected water for backwashing purposes;
* 700 m^3 as contact chamber;
* 3,900 m^3 as useful storage for local clear water distribution and clear water transport to the WK-Plein.

Since the electricity company of Suriname cannot supply the required power, power generation facilities will be provided at the new WTP, which will also supply the required power to the well-field.

The backwash water will be pumped into a sedimentation basin, located close by the new WTP. The effluent of this sedimentation basin will be drained locally.

1.3.3 CLEAR WATER TRANSMISSION MAIN: VAN HATTEMWEG - WK-PLEIN

For the transport of clear water to the main distribution centre at WK-Plein, a ϕ 800 mm steel transmission main with a total length of 16,800 m will be laid, mainly along the Indira Ghandiweg and crossing the Saramacca canal by means of an underwater river crossing. Pumps will be installed in the CWR of the new WTP. Water hammer protection will be realized by means of 2 parallel connected airvessels of 75 m^3 each.

1.3.4 DISTRIBUTION CENTRE WK-PLEIN

At the existing SWM location at the WK-Plein, a new Clear Water Reservoir of 8,000 m^3 will be provided, bringing the total storage capacity at WK-Plein at 10,000 m^3. From this clear water reservoir water will be partly supplied directly into the distribution network and partly conveyed to the Blauwgrond reservoir complex in the north. For these purposes adequate new pumping facilities will be provided on top of this CWR. A central control room will be located here, where information of all production units and pumping stations will be displayed. Furthermore a new central laboratory has been foreseen on this location as well.

1.3.5 CLEAR WATER TRANSMISSION MAIN: WK-PLEIN - BLAUWGROND

In order to safeguard the water supply in the northern part of the city, water will be conveyed through a 7,500 m long, ϕ 500 mm steel transmission main to the Blauwgrond reservoir complex. Water hammer protection will be realized by means of an air vessel of 45 m^3.

1.3.6 DISTRIBUTION SYSTEM

For the upgrading c.q. the extension of the distribution network a total of 53 km of pipe, ranging in diameter from ϕ 200 mm to ϕ 600 mm, will be laid. Upgrading and extensions are foreseen both north and south of the Saramacca canal. Furthermore equipment and materials for the rehabilitation of the existing network will be provided, as well as equipment and materials for block metering.

1.3.7 MISCELLANEOUS PUMPING SYSTEMS

In order to meet the required flows and pressures in the new configuration of the distribution network, the pumping facilities at Republiek, Livorno and Blauwgrond will be replaced. Furthermore a booster station will be installed in the old Republiek main, between Republiek and the Clear Water Reservoir of the new WTP.

1.3.8 REHABILITATION WORKS REPUBLIEK

The treatment plant at Republiek will be modified drastically. A new aeration chamber will be constructed to replace the old one, which is too small and in a very poor state.
To facilitate adequate backwashing, without making the backwash flow and facilities too large, the sand filters will be split into units of 30 m^2 each. To make backwashing with air possible, the filter bottoms will be replaced by false bottoms provided with nozzles. In order to improve the efficiency and to obtain longer filterruns, the height of the filterbed of the filters and the height of the side walls will be increased.
The piping gallery between the sand filters and the shell filters will be replaced completely.
Backwash facilities, consisting of backwash pumps and air blowers, complete with piping system will be installed.
To improve the contact time and lengthen the filter-run of the shell filters, the filter-boxes will be heightened.
Disinfection facilities will be provided at the existing clear water reservoir.

1.3.9 REHABILITATION WORKS WK-PLEIN

In general, rehabilitation works at the WK-Plein treatment plant will be realized along the same line as described above for the Republiek treatment plant. Here the existing aeration chamber will upgraded.

1.3.10 REHABILITATION WORKS LIVORNO

At Livorno, the rehabilitation works consist mainly of replacement of corroded pipework. The addition of shell filters for conditioning of the water has been foreseen, as well as the construction of a new clear water reservoir.

1.4 REPORTING

The Inception Report of the Detailed Design phase of the project was submitted in June 1990. The Mid Term Report, which already covered all aspects of the present Detailed Design phase, was issued in 1991 and was discussed in the IADB-office in Washington on the 3rd and 4th of April. The Draft Final Report was submitted early September 1991 and subsequently discussed with the client in The Netherlands at the end of October.

Besides these main reports and the present Final Report, the Consultants has issued a number of Special Reports, each covering a specific subject. These Special Reports provided timely information to all concerned parties with regard to investigations, specific developments, prevailing ideas, possibilities and conclusions. Because some of these reports have a temporary character, all relevant data are again integrated in this Final Report.

1.5 ACKNOWLEDGEMENT

During the initial stage of the project, when data were collected, during the further implementation of the study and during the preparation of the financial and economic analyses, the Consultants have received valuable assistance and co-operation from many authorities, institutions and from the management and staff of the SWM.

In this respect, special mentioning should be made of the Directors and staff of the SWM for their continuous cooperation, guidance and the fruitful discussions, without which it would have been impossible to achieve the results as reflected in this Final Report.

The Consultants wish to express their gratitude for the constructive cooperation in this way. The Consultants furthermore wish the SWM success with the negotiations for the financing of the envisaged project and hope that the implementation phase will start in the near future.

2 WATER DEMAND PROJECTIONS

2.1 INTRODUCTION

At the start of the study, the socio-economic situation of Suriname has been evaluated. The basis for this evaluation comprised of the socio-economic survey, held during the Feasibility Study, recent data from the SWM administration and additional data gathered from other institutions, both governmental as well as non-governmental. The results were previously presented in Special Report 1: Water Demand Projections, which was issued in August 1990 and have been incorporated in Volume 2 of this Final Report as chapter 2. A summary of the conclusions, completed with the overall production requirements have been presented in this chapter.

2.2 MACRO-ECONOMIC BACKGROUND: TREND AND PROSPECTS

Projection of water demand over a twenty year period, - even under the best of circumstances an activity beset with uncertainties - is subject to unusual additional difficulties in the case of Paramaribo. This relates to the adverse economic developments in Suriname during the 1980's and the lack of clear directions in economic policy as well as to the relative dearth of reliable data, particularly as far as population growth and spatial development are concerned.

Suriname's GDP growth was adversely affected in the 1980's by trends in prices for its major export products, bauxite, alumina and aluminum, by the suspension of assistance from The Netherlands in 1982 and by continuing hostilities in the country's interior. As a result the GDP in 1987 had declined to the 1975 independence level. In 1988 and 1989 external conditions improved somewhat and a positive GDP growth was registered during both years. Over the period 1980-1989 a compound GDP decline of 0.2% per annum was the net result. The GDP composition changed markedly over the period, with Government, mining, transport and banking sectors growing in importance at the expense of the manufacturing, construction and commercial sectors. The yearly inflation averaged 13.6% during 1980-1989 with a peak of 53.4% in 1987 - the disaster year for the Suriname economy - when Government recognition of parallel market prices and price increases for Government controlled staple goods coincided. Suriname's external position weakened considerably over the period 1980-1989 with continuing balance of payments deficits throughout the period. Acute shortages of foreign exchange resulted, adversely affecting output, prices and the structure of the economy. The official exchange rate of Sfl 1.78 to US$ 1.00 was maintained throughout this period, but a rapidly diverging parallel exchange rate developed. The Government's fiscal deficit widened substantially over the period. While no accurate estimates are available, it is widely believed that unemployment is in the range of about one third of the labour force. Simultaneously, substantial net out migration has i.a. drained the country of skilled labour.

The adjustment process implicit in the above developments has been particularly painful for the unemployed and those in fixed wages employment. The Government has recognised the need for a short term recovery and adjustment programme and a medium term development programme. The short term adjustment programme is currently being formulated with EEC assistance and will most likely comprise measures to reduce external and fiscal deficits and reduce the monetary overhang, including a foreign exchange rate adjustment, as well as providing for substantial short term balance of payment support by the donor community and for social support measures to shield the poorer segment of the population from the adverse consequences of adjustment. The existing multi year development programme requires updating and modification. It is as yet unclear to what extent both short and medium term plans will be supported by all sections of Suriname's diverse community.

In view of the unfavourable recent economic trends and the uncertainties around the short term adjustment programme and the medium term development plan, two scenarios are used for the purpose of water demand projections:

* **Scenario A (optimistic)**
  this scenario assumes the early implementation of an endorsed adjustment programme and multi year development programme and hence the quick resumption of substantial aid flows. In this scenario a 3% real GDP growth and a 5% inflation per year are assumed.

* **Scenario B (pessimistic)**
  this scenario assumes the continuation of present trends and policies, a very modest annual real GDP growth of 0.5% and a continuing high inflation rate of 10% per year.

2.3 DEMOGRAPHIC AND SPATIAL TRENDS

Estimation of demographic and spatial trends are hampered by lack of reliable data since 1980 when the last population census was conducted. This is particularly unfortunate in view of the importance of external migration, which is only partially covered in projections based on civil registration. Additionally, hostilities in Suriname's interior have led to refugee movements both to Paramaribo and across the border into French Guyana.

Suriname's population (taking into account emigration to The Netherlands as far as captured by The Netherlands' Central Bureau of Statistics) was estimated at 399,000 at the end of 1988, with a net population growth of 1.34% per year during 1981-1988 as compared to a natural yearly population growth of 2.16% during the same period.

For the purpose of population projections, the latter percentage (assuming restoration of the migratory balance from 1991 onwards) has been assumed for the optimistic scenario A. The lower percentage, assuming continuing net emigration, has been applied for the pessimistic scenario B.

Population estimates for urban Paramaribo are not available. Information is limited to data on Paramaribo district and the adjacent Wanica district population, which are estimated at 214,400 and 73,400 respectively by the end of 1990, including an estimated refugee population of 12,200. The population of the two districts combined comprises about 70.2% of Suriname's estimated 1990 population. In view of this, it has been assumed that population in these districts will grow at the same rates as national population growth during the projection period under the two scenarios.

There is no effective physical planning framework as yet to guide urban development in Paramaribo, although the 1972 town planning act provides the legal basis for this. In view of this and the relative lack of quantitative information, recent trends can provide only an indication of patterns of spatial development that can be expected in future. Population densities in Paramaribo (based on 1980 census data) are low by international standards. Before 1980, urban development was characterised by low density extensive growth. In the 1980's this process gradually slowed down as reflected in private sub-division approvals, building permits issued and the completion of Government low income housing projects. Paramaribo's population increase during this period has primarily resulted in increasing densities in existing built-up areas, particularly in the relatively poorer southern area of the city. Outward expansion has been very limited, except in the south-western part of Paramaribo district.

The SWM service area of about 175 km² includes most of Paramaribo district and part of the adjacent Wanica district, including a narrow corridor along the road to Zanderij airport situated in Para district. The airport area itself is also included in the SWM service area. Current population of the SWM service area and its spatial distribution has been estimated at about 236,500 (end 1989) based on SWM connections data by billing areas, information on non-covered households (estimated at 6.3% of households within the area) and a household size of 4.78 persons (1980 census). Based on current densities on both sides of the present service area boundary, there appears to be no justification at present to extend the SWM service area significantly, except to include the remaining past of south-west Paramaribo district where further outward expansion is envisaged. Population of the SWM service area is expected to grow at the same rates as Paramaribo and Wanica districts under the two scenarios and is expected tot reach 375,200 in 2010 under scenario A and 319,300 under scenario B. Expansion of the SWM network within the current service area will de facto increase the area coverage by the year 2000 from the present 104 km² to 116 km² (scenario A) or 112 km² (scenario B) according to a population allocation model applied to both scenarios. This provides indicative population densities per SWM billing area, with density increases slightly higher for scenario A (where more rapid population growth more than offsets the more extensive pattern of development) than for scenario B.

2.4 RESIDENTIAL DEMAND

Historical trends and patterns of consumption by tariff group combined with the results of a 1987 sample survey of households in Paramaribo, Wanica and Para districts have been combined in estimating characteristics of water demand. Average daily per capita water consumption was 126.8 litres in 1989. Socio-economic characteristics of water users and estimates of price and income elasticities of demand have been calculated, based on the survey data. The relatively high incidence of poverty is likely affected by people reporting less than their actual income. The survey found that only 6.3% of households in Paramaribo district had an exclusive private water supply (wells, rainwater collection) and was not connected to piped supply. In the semi-rural district Wanica, this percentage was much higher. Intermittent SWM supply affected 13.3% of the households (mostly up to six hours a day). 60.2% of the surveyed households experienced low pressure (but again, not generally for more than six hours a day). Based on supply received by households experiencing these problems as compared to non-problem households, it is estimated that supply constraints to aggregate residential water consumption amounted to 5.6% of metered residential consumption in 1989. Price elasticities of water demand found, ranged from -0.623 to -0.654 and income elasticities from 0.076 to 0.10, in line with findings elsewhere in Latin America. With the above findings current residential water demand has been estimated and projected forward for the years 2000 and 2010 (see table 2.1).

2.5 NON-RESIDENTIAL DEMAND

A distinction has been made between industrial, commercial, institutional, and miscellaneous demand, which were analyzed and projected separately based on historical trends and patterns of consumption (including spatial patterns). Industrial and commercial water consumption are heavily dominated by a relatively small number of large customers (for example, 5 connections with more than 5,000 m^3 water consumption per month accounted for 23.9% of the total industrial c.q. commercial water consumption in 1989). In order to improve the understanding concerning patterns of water consumption, a small sample survey was carried out among large industrial water consumers. Based on a consumer typology developed, it was attempted to explain water consumption patterns separately for each group within this typology. Regression fits were found to be unsatisfactory. However, the qualitative survey results strongly indicated a link between industrial water demand and output/turnover (and the predominant foreign exchange constraints thereon). Hence industrial and commercial water demand was assumed to grow proportionally with the real GDP growth under both scenarios (see table 2.1). The survey clearly indicated insensitivity of demand for price increases, therefore no price influence was assumed.

A similar domination of large customers was found for institutional water consumption. Also for this group, no adequate explanatory regression fits could be found and for the projections it was therefore assumed that this type of demand will grow proportionally with the population increase (see table 2.1). Miscellaneous water consumption consisting of temporary (construction) and shipping connections, comprises a very small segment of water consumption. In the projections it has been assumed to grow proportionally with the real GDP.

2.6 TOTAL DEMAND

Projections for total water demand and its constituent parts are presented in table 2.1 for scenarios A and B.

Total water demand in the year 2000 is 38.3% higher than the present (1989) water consumption under scenario A and 23.3% higher under scenario B. By the year 2010 these percentages are 74.3% and 38.6% respectively.

As expected, results are quite sensitive to differences in population projections. As residential demand is very dominant in the total demand, 73.6% to 76.5% in the year 2010 under scenarios A and B respectively, overall demand projections are heavily affected by residential water demand sensitivity to price elasticity assumptions.

Table 2.1: Total water demand (per year)

Year	2000		2010	
	*1,000 m^3	%	*1,000 m^3	%
SCENARIO A (optimistic)				
- *residential demand*	14,024	74.3%	17,497	73.6%
- industrial and commercial demand	2,602	13.8%	3,496	14.7%
- institutional demand	2,186	11.6%	2,707	11.4%
- temporary and shipping demand	65	0.3%	88	0.4%
TOTAL	18,877	100.0%	23,788	100.0%
SCENARIO B (pessimistic)				
- residential demand	12,774	75.9%	14,481	76.5%
- industrial and commercial demand	1,986	11.8%	2,087	11.0%
- institutional demand	2,017	12.0%	2,304	12.2%
- temporary and shipping demand	50	0.3%	52	0.3%
TOTAL	16,827	100.0%	18,926	100.0%

2.7 OVERALL PRODUCTION REQUIREMENTS

The total required production capacity is not only determined by the total demand. Factors such as the maximum day, unaccounted for water and the internal water use at the treatment plants will have to be taken into account as well. In table 2.2 the various influencing factors have been presented, including their development in the time.

Due to the fact that no main production and distribution meters are installed, the actual daily quantity of distributed water and the frequency distribution thereof is not known. Records which are kept by the SWM are only rough estimates. Therefore prevailing peak factors, hourly as well as daily, cannot be determined. Furthermore, the prevailing production capacity of the SWM is insufficient to cover the actual demand. Consequently, any peak factor found under these restricted conditions will not reflect the future conditions when the new installation will have become operational. Therefore, the maximum day factor has been selected arbitrarily, based on experience with other water supply enterprises. In the following required production estimates, a maximum day factor of 1.2 has been taken into account.

? see p. 7 5.6%

Table 2.2: *Factors determining the overall production capacity*

	1990	1991	1992	1993	1994	1995	---->	2010
Maximum day factor	20.0%	20.0%	20.0%	20.0%	20.0%	20.0%	---->	20.0%
Net unaccounted for water	25.5%	24.4%	23.3%	22.2%	21.1%	20.0%	---->	20.0%
Internal use water treatment plants	5.0%	5.0%	5.0%	5.0%	5.0%	5.0%	---->	5.0%

The first measurements performed in the framework of this study indicated a percentage of Unaccounted for Water of 29%. A correction for unmetered supply has been applied for the percentage of Unaccounted for Water, since the correction for unmetered supply was made with the calculations of the Water Demand Projections. It is anticipated that an unaccounted for reduction programme will reduce the quantity of unaccounted for water gradually in the coming years. A minimum level of 20% has been assumed, which is considered to be acceptable in other countries under similar conditions. This level of 20% is assumed to be reached in 5 years time. Furthermore, it is anticipated that the unmetered connections will be phased out in the coming 5 years and that all house connections will have been provided with a properly functioning water meter.

The internal water use at the treatment plants is estimated to amount to 5% of the total quantity of water produced. This water is used for filter backwashing and general purposes.

Based on the demand projections as presented in table 2.1 and the criteria presented in table 2.2, the projection of the required overall water production capacity can be made. They are presented in tables 2.3 and 2.4 and graphically in figure 2.1.

Table 2.3: Overall production requirements in m^3/hr: Scenario A

Description	1990	1995	2000	2005	2010
Total demand	1,843	1,920	2,155	2,419	2,716
Additional for maximum day	369	384	431	484	543
Total demand maximum day	2,212	2,304	2,586	2,903	3,259
Unaccounted for water	757	576	646	726	815
Total to be distributed	2,969	2,880	3,232	3,628	4,073
Internal water use	156	152	170	191	214
Overall production requirements	3,126	3,032	3,403	3,819	4,288

Table 2.4: Overall production requirements in m^3/hr: Scenario B

Description	1990	1995	2000	2005	2010
Total demand	1,835	1,812	1,921	2,037	2,160
Additional for maximum day	367	362	384	407	432
Total demand maximum day	2,202	2,174	2,305	2,444	2,592
Unaccounted for water	754	543	576	611	648
Total to be distributed	2,955	2,717	2,881	3,055	3,241
Internal water use	156	143	152	161	171
Overall production requirements	3,111	2,860	3,033	3,216	3,411

3 EVALUATION EXISTING WATER TREATMENT PLANTS

3.1 INTRODUCTION

In chapter 2 the overall required production capacity up to the year 2010 has been determined, based on demand analyses and on influencing factors such as maximum day, unaccounted for water and the internal water use at the treatment plants. Before the additional production capacity can be determined, the existing production capacity, suitable for further long term use will have to be established.
In Special Report 3: Existing Water Treatment Plants, which was issued in November 1990 and in chapter 3 of Volume 2 of this Final Report, a technical evaluation of the existing treatment plants has been made in order to assess the possibilities and the feasibility of the rehabilitation of these facilities in order to make them suitable for a continued operation. A summary has been presented in this chapter.

In the present water supply system of Paramaribo, there are 7 production units (well-fields + treatment plants), see table 3.1. The three most recent plants, Lelydorp, Flora and Benie were taken into operation in recent years to alleviate the shortcomings of the production capacity and are consequently of a temporary character. They are constructed of steel containers. When the new production facility comes into operation, these three plants will be shut down.
Therefore, these three plants are not included in the evaluation and the recommendations for rehabilitation. An exception is made for Lelydorp. This plant is taken into account in the evaluation, because it is located close to the planned location of the new plant, and abstracts water from the same aquifer.

Table 3.1: Existing water treatment plants

Treatment plant	In operation since	Capacity (m^3/h)
Republiek	1932	400
WK-plein	1958	700
Leysweg	1972	525
Livorno	1980	450
Lelydorp	1985	250
Flora	1989	100
Benie	1990	50
Total		2,475

Because of the present shortage of drinking water, all facilities are operated at maximum capacity, or even at an overload (especially Leysweg and to a lesser extent, Lelydorp). The possible increase of the production of the existing facilities is only slight.

3.2 EVALUATION

3.2.1 Quality Aspects

In Tables 3.2 and 3.3 the process parameters and the performance of the treatment plants are summarized.

Table 3.2: Summary process parameters treatment plants

Parameter	Unit	Treatment plant Republiek	WK-plein	Livorno	Leysweg	Lelydorp
Flow Q	m^3/hr	400	700	460	525	300
Treatment system		1	1	2	2	1
Aeration						
- type	-	nozzles	nozzles	tower	nozzles	tower
- flow per nozzle	m^3/hr	2.6	4.6		7.3	
- falling height	m	0.5-1.5	2	5	1-2	2-3
- surface load	m/hr	8.0	5.8	46.9	7.2	75.0
Sand filtration						
- type	-	open downflow	open downflow	pressure downflow	open downflow	open downflow
- height filter bed	m	0.4-0.5	1.5-2	2	1.55	0.7-1.3
- grainsize filterbed	mm	1-2	1.5-2.8	1.4-2.8	2-4 (1-2)	1-2
- filtration rate	m/hr	2.3	3.9	9.6	7.2	7.2
Backwashing						
- frequency	-	1/day	1/day	1/day	2/day	1/day
- backwash water rate	m/h	3.5	9.9	31	10.9	??
- backwash air rate	Nm/hr	n.a.	44	69	??	??
- backwash time	min.	5	20	10	30	20-30
Shell filtration						
- type	-	open 1 upflow	open downflow			open downflow
- filtration rate	m/hr	3.9	6.4			11.2
- minimum contact time	min.	7	6			3
Clear water reservoir						
- number of reservoirs	-	2	3	2	1	1
- total volume	m^3	4,400	3,750	1,440	4,000	1,000

Legend:
Treatment system
1. aeration/sand filtration/aeration
2. aeration/sand filtration/aeration/sand filtration

With regard to the removal of iron, all plants perform well. The results of Lelydorp show a large fluctuation, which could point at breakthrough of the filters.

The aggressivity (Ph, SI-index) of the effluent of the plants with a shell filter (Republiek, WK-plein and Lelydorp) is considerably lower than that of the plants without (Leysweg and Livorno). It can be concluded that the shell filtration is indispensable. In general, the contact time of the existing shell filters appears to be too short, because of short circuits and too late refilling of the filterbed.

Table 3.3 Summary performance treatment plants

Parameter	Unit	Republiek a.	Republiek b.	WK-plein a.	WK-plein b.	Leysweg a.	Leysweg b.	Livorno a.	Livorno b.	Lelydorp a.	Lelydorp b.
Treatment system		1		1		2		2		1	
Acidity (Ph)	-	5.3	7.5	6.3	7.9	5.9	6.4	5.9	6.7	6	7.3
Carbon dioxide (CO_2)	mg/l	60	8	110	24	166	28	98	18	96	40
Bicarbonate (HCO_3)	mg/l	11	87	138	140	76	104	92	67	79	98
SI-index	-	-4.7	-0.6	-2.0	-0.1	-2.3	-1.7	-2.8	-2.1	-2.5	-0.8
Chloride (Cl-)	mg/l	10	13	284	284	288	291.0	124	124	32	32
Iron (Fe(tot))	mg/l	2.0	0.1	5.2	0.1	13.1	0.2	4.6	0	3.5	1.2
Manganese (Mn^{2+})	mg/l	0.4	0.1	0.5	0.1	2.1	1	0.2	0	1.1	0.3
Hardness (Mg_2^+/Ca_2^+)	oG	0.9	4.1	6.4	8.2	15.5	14.9	4.6	3.4	3	4.3
Ammonium (NH_4^+)	mg/l	0.4	0.2	0.9	0.1	1.3	0.9	0.4	0.1	0.6	0.2
Oxygen (O_2)	mg/l	1.5	7.4	0.8	1.3	0.8	2.8	1.8	5.1	1	5.9

Legend:
a. raw water
b. clear water

Treatment system:
1. aeration/sand filtration/shell filtration
2. aeration/sand filtration/aeration/sand filtration

At Leysweg and Lelydorp the effluent does not meet the WHO-standard for manganese. At Lelydorp the value after sand filtration is much better, indicating a possible error in analyzing. However, the regular SWM analyses show the removal of iron is satisfactory. At Leysweg, all measured values are high. Also the operational problems with the second filtration step indicate an overloading of the system.

The ammonium guideline for the EEC is exceeded by all the treatment plants. The maximum acceptable level is exceeded only by Leysweg. This is objectionable because of the high oxygen demand of ammonium, and therefore because of the risk of anaerobic water in the distribution system. The WHO has no guideline value for ammonium, indicating that there is no health risk involved.

The buffering capacity of the effluent of the treatment plants is acceptable, though it does not reach the value recommended by KIWA.

WK-plein and Leysweg are faced with salt water intrusion. The chloride content exceeds the WHO guideline.

The bacteriological quality of the effluent of all treatment plants is satisfactory, according to the analyses performed.

3.2.2 Operational Aspects

All the aeration systems applied function reasonably well. At most of the treatment stations there are problems with the aggressivity of the water causing corrosion and pollution of the aeration system, especially at Livorno and Republiek.

The sand filters are responsible for most of the operational problems at the treatment plants. With the exception of Livorno all plants have clogged filterbeds, resulting in the risk of breakthrough of iron, and short filterruns. This is caused by poor backwashing: the backwash rate is too low and in some cases also the backwash time is insufficient. At Leysweg the problem is most pressing, because of the high iron concentration in the raw water, and the high filterrate. The other plants perform reasonably well, because of the low iron concentration and a low filtration rate.
A second problem, occurring at most sand filters, is the absence of a weir at the effluent side. With a clean filterbed, after backwashing the resistance is so low, that the filterbed can run dry. This

causes poor distribution of the influent over the filter, which may cause the iron to penetrate deeper into the filterbed, giving an increased risk of breakthrough of iron and of clogging of the filterbottom.

In the shell filters the filterbed is not always refilled in time, resulting in short contact times. This is partly caused by logistical problems concerning the supply of shells. Furthermore, in some filters the filterbed runs dry, further shortening the effective contact time considerably. Lastly, the distribution of shells is often unequal as well as the distribution of the water over the shell surface, caused by inadequate inlet of the water.

3.3 OPTIMUM PROCESS

Based on the evaluation of the existing treatment plants and literature, the optimum system for the treatment of the groundwater around Paramaribo, can be determined. The optimum process consists of the following elements:

1. aeration;
2. sand filtration;
3. shell filtration;
4. disinfection;
5. wash water reservoir;
6. clear water reservoir.

Table 3.4: Optimal process parameters

Parameter	Unit	new WTP
Flow Q	m^3/hr	2,000
Treatment system		aeration/sand filtration/ shell filtration/disinfection
Aeration		
- type	-	nozzles
- flow per nozzle	m^3/hr	5.0
- falling height	m	2
- surface load	m/hr	7.5
Sand filtration		
- type	-	open downflow
- height filter bed	m	1.5
- grainsize filterbed	mm	1 - 2
- filtration rate	m/hr	5.0
Backwashing		
- frequency	-	0.5/day
- backwash water rate	m/hr	30 - 40
- backwash air rate	Nm/hr	60
- backwash time	min.	15
Shell filtration		
- type	-	open downflow
- filtration rate	m/hr	10.0
- minimum contact time	min.	5
- average contact time	min.	8
Disinfection		
- disinfectant	-	sodium hypochlorite
- dose	mg/l	0.5 - 1
- contact time	min.	15

It is the system, applied at most existing treatment plants, with disinfection added to it, as a safety measure against recontamination in the transport and distribution system. In the feasibility study, possible disinfection systems have been evaluated. It is concluded that the use of sodium hypochlorite is most feasible, mainly because it is locally available. A separate washwater reservoir with filtered, not disinfected water is required: the removal of ammonium by sand filters is a bacteriological process; the use of disinfected water would kill this process.

In table 3.4 the optimal process parameters are given for each step. Compared with the actual parameters of the existing plants, the following remarks can be made:

- The filtration rate of the sand filters is higher than the Republiek and WK-plein; this is allowed, if proper backwashing is taken care of.
- The backwash rate is considerably higher, to ensure efficient backwashing and prevent permanent clogging of the filterbed.
- The effective contact time is somewhat longer than in the existing plants, to improve the decrease in aggressivity of the water and increase the buffering capacity.

3.4 REHABILITATION

The main rehabilitation works, to operate each of the plants optimally are:

REPUBLIEK
The aeration chamber has to be replaced by a new one. The sand filters have to be split into 6 smaller ones, and the height of the walls of the filter will be increased to improve performance and facilitate adequate backwashing. The piping gallery will have to be replaced to facilitate backwashing.
The height of the walls of the shell filters has to be increased; the one upflow filter has to be rebuilt into a downflow filter.

WK-PLEIN
As with Republiek, the height of the sand filters has to be increased, and the piping gallery has to be replaced, to facilitate adequate backwashing.
To lower the chloride content of the drinking water, produced by WK-plein, this effluent is proposed to be blended in the clear water reservoir with the fresh water from the new treatment plant.
At WK-plein a large new clear water reservoir is required, because WK-plein will be the centre of the distribution system for Paramaribo.
For the same reason, the Consultants propose to built a new control room for the operation of the whole water supply system at WK-plein.

LEYSWEG
At Leysweg the problems with the operation of the treatment plant and the quality of the effluent are most pressing. Three alternatives have been evaluated:

alt.1 operating Leysweg at 500 m^3/hr, blending of the water at WK-plein;
alt.2 operating Leysweg at 250 m^3/hr, blending of the water at WK-plein, increasing the production capacity of the new treatment plant with 250 m^3/hr;
alt.3 taking Leysweg out of operation, increasing the production capacity of the new treatment plant with 500 m^3/hr.

The alternatives have been evaluated on the quality and the costs of the drinking water, see table 3.5. It is clear that alternative 3 is optimal, as it scores best on quality as well as costs. The Consultants therefore recommend to take Leysweg out of operation as soon as the new treatment plant is in operation and the rehabilitation works are finished.

LIVORNO
The main rehabilitation works at Livorno are the construction of shell filters and of a new clear water reservoir. Air mixers and some piping have to be replaced because of corrosion. Furthermore, maintenance is required to prevent further problems with corrosion.

Table 3.5 Evaluation alternatives Leysweg

Criterium	Alternative 1	Alternative 2	Alternative 3
Quality aspects	not	o	+
Cost aspects	feasible	-	+

DISINFECTION AT ALL FACILITIES

Although the water distributed by the SWM is bacteriologically safe, disinfection has to be added at the plants, as a safety precaution.
The Consultants recommend disinfection by means of sodium hypochlorite, mainly because this is locally produced, and therefore readily available.

3.5 PHASING OF THE REHABILITATION WORKS

For some of the rehabilitation works on Republiek and WK-plein, the treatment plants have to be taken out of operation. Because of the present shortage of drinking water, this is unacceptable as long as the new plant is not in production yet. Implementation of those works will have to wait until 1994/1995.
Most of the works however, can (or even have to) be implemented before the new treatment plant is finished.

4 NEW WATER TREATMENT PLANT

4.1 INTRODUCTION

In chapter 2 the overall required production capacity up to the year 2010 has been determined and in chapter 3 the existing treatment plants have been evaluated. By combining these two data sets the capacity of the new Water Treatment Plant (WTP) can be determined.
In Special Report 4: New Water Treatment Plant, which was issued in November 1990 and in chapter 4 of Volume 2 of this Final Report, the optimum treatment capacity for phase I has been determined. Furthermore a set of design criteria has been prepared, based on the evaluation of the performance of the existing treatment plants and on general process parameters. Finally, the future operation of this new treatment plant on its own and in relation to the existing facilities has been considered. A summary has been presented in this chapter.

4.2 RAW WATER QUALITY AND SELECTION TREATMENT PROCESS

The new treatment plant will be located in the Rijsdijk/Lelydorp area, along the van Hattem road approximately 800 to 1,000 m from the Pad van Wanica, the main road between Paramaribo and the international airport A. Pengel. The expected raw water quality, based on a number of analyses, is given in Table 4.1. The main parameters that need improvement are aggressivity (Ph, SI-index), iron, manganese and ammonium.

Table 4.1: Expected raw water quality, macro parameters

Parameter	Unit	Expected value
Acidity (Ph)	-	5.5 - 6.5
Carbon dioxide (CO_2)	mg/l	100 - 120
Bicarbonate (HCO_3^-)	mg/l	100 - 120
SI-index	-	-2 à -3
Chloride (Cl-)	mg/l	50 - 60
Iron (Fe(tot))	mg/l	5 - 7
Manganese (Mn^{2+})	mg/l	0 - 0.5
Ammonium (NH_4^+)	mg/l	0 - 0.5

Based on the experience of the SWM and the pilot plant tests performed during the Feasibility Study, the following treatment process has been selected: aeration, sand filtration, shell filtration and disinfection.
In Table 4.2 the main process parameters are given, based on the evaluation of the existing treatment plants in Special Report 3, summarized in chapter 3.

Table 4.2: Expected raw water quality, heavy metals

Parameter	Unit	Expected value
Arsenic (As)	µg/l	< 2.0
Cadmium (Cd)	µg/l	< 0.50
Chromium (Cr)	µg/l	< 2.0
Copper (Cu)	µg/l	< 2.0
Lead (Pb)	µg/l	< 5.0
Nickel (Ni)	µg/l	< 10
Zinc (Zn)	µg/l	< 100
Mercury (Hg)	µg/l	< 0.2

4.3 OPTIMUM PHASE I CAPACITY

The optimum capacity of phase I of the new WTP depends on the development of the water demand. This development is unknown, because the economic growth is very uncertain. For this reason, a financial analysis (calculation of the NPV of various alternatives) has been performed, to determine the capacity with the minimal financial risk. The conclusion is that the optimum capacity is 2,000 m^3/hr, with future extensions of about 350 - 650 m^3/hr, depending on the actual economic growth.

4.4 DESCRIPTION OF THE TREATMENT PROCESS

The aeration takes place in an aeration chamber, standing apart from the filtration building. A system with natural ventilation has been chosen, to avoid the use of vulnerable mechanical equipment and the risk of clogging.

The sand filtration unit consists of 8 open, concrete filters. The filters are operated as downflow, constant rate filters, at a filtration rate of 5 m/hr. The filter medium is sand, 1-2 mm, with a bed thickness of 1.5 m.

A total of 4 shell filters have been foreseen. The filter boxes are identical to those of the sand filters. The filtration rate is 10 m/hr and the minimum required contact time is 5 minutes, resulting in a minimum bed thickness of 0.85 m. The maximum bed thickness is 2.0 m, which means that the filters have to be refilled every 2 - 3 months.

Disinfection of the clear water will be carried out using sodium hypochlorite, as this is locally produced and readily available. The solution will be injected at the inlet weir of the clear water reservoir. The turbulence of the water ensures adequate mixing. The required contact time of 15 minutes is guaranteed by maintaining a minimum water level of about 0.5 m in the clear water reservoir.

Both sand and shell filters can be backwashed. Sand filters with water and air, shell filters with water only. The required backwash rate is 30 m/hr in both cases. For backwashing, not-disinfected water has to be used, to protect the biological processes in the sand filters (ammonium removal). Therefore, the washwater is abstracted from a separate part of the clear water reservoir.
One of the conclusions of the Environmental Impact study is that there is no health hazard if the backwash water is discharged into surface water, and if the base flow of the surface water is large enough, the effects on the aquatic life and the aesthetical problems are minimal.

The Consultants recommend however, to construct an equalizing/sedimentation basin of about 2,000 m^2. The clarified effluent of this sedimentation basin can be drained to the nearby open water.

The total required volume of the clear water reservoir is about 6,000 m^3:

- 1,500 m^3 of not-disinfected water for backwashing;
- 650 - 700 m^3 as contact chamber;
- 3,900 m^3 useful storage for balancing production and clear water pumping.

Besides the water produced by the new WTP, the bulk of the water produced by the existing Republiek water treatment plant, which is scheduled to be rehabilitated, will be led to this clear water reservoir. The water from this clear water reservoir is partly used for distribution to the northern part of the Pad van Wanica and partly for transport to main distribution centre at WK-plein. For both purposes, separate clear water pumps will be available at the new WTP.

The facilities present at the WTP are: power house, disinfectant storage and dosing building, office, control room, a building for the backwash blowers and a general purpose building.
Most facilities are adequate for a future extension as well.

4.5 OVERALL WATER SUPPLY SYSTEM

The total water supply system for Paramaribo after construction of the new WTP and rehabilitation of the existing Republiek, WK-plein and Livorno plants, will thus consist of four production and distribution stations and one distribution station, see table 4.3.
The total production capacity amounts to 3,550 m^3/hr.

Table 4.3: Future water supply system Paramaribo

Station	Production (m^3/hr)	Distribution	Transmission
Republiek	400	x	
WK-plein	700	x	x
Livorno	450	x	x
New WTP	2,000	x	x
Blauwgrond		x	

The operation of the overall system is concentrated at a central control room, to be constructed at WK-plein. There, all the required main data to manage the entire production and distribution system are collected. These data are:

- actual production at all production units;
- level in all clear water reservoirs;
- actual distribution flows;
- actual transmission flows.

It is recommended to use radio telecommunication for the transmission of the data, because the SWM has a reasonably good working radio network already, which can adopted for this purpose at relatively low cost.
More detailed information with regard to the control systems have been presented in Special Report 12: Pump Selections, Power Supply and Control Systems, issued in January 1991 and in chapter 14 of Volume 2 of this Final Report, of which a summary has been presented in chapter 14 of this summary.

5 WATER RESOURCES INVESTIGATION

5.1 INTRODUCTION

Considering the complexity of the present project, whereby components on feasibility level, among which hydrogeological investigations and the subsequent source selection, had to be implemented simultaneously with the preparation of preliminary detailed designs, the necessity of the timely taking of decisions was imperative. In order to obtain as early as possible an indication of where the Phase I well-field would be located, a preliminary selection was made, based on a sound technical and financial evaluation. This preliminary selection has been discussed in Special Report 2: Source Selection, issued in October 1990.

The results of the field investigations and the mathematical models of the Zanderij and Coesewijne aquifers subsequently confirmed this preliminary selection. In Special Report 9: Mathematical Model of the Zanderij and Coesewijne Aquifers, issued in January 1991 and in Annex 5 of this Final Report, the regional mathematical model has been discussed. Special Report 10: Water Resources, also issued in January 1991 and incorporated as chapter 5 of Volume 2 of this Final Report, deals with the overall aspects of water resources and discusses the analytical and the hydraulic models used for the more detailed investigations of the prospective well-field.
In this chapter a summary of the conclusions of the above mentioned three special reports is presented.
Closely related and as far as a number of topics is concerned more or less integrated with the water resources study, is the environmental impact assessment. This matter has been dealt with separately in Special Report 5 -and in chapter 7 of Volume 2 of this Final Report- since other topics, not directly related to the water resources investigation are discussed as well.
A summary of Special Report 5 has been presented in chapter 7 of this summary.

5.2 HYDROGEOLOGICAL INVESTIGATIONS

During the period from August 1990 until January 1991 a water resources study has been carried out around Paramaribo city in Suriname. The water resources study was carried out by a team of hydrogeologists and environmentalists of IWACO. The study was focused on groundwater resources in the Zanderij and Coesewijne aquifers in an area of 1,000 km^2 in the coastal plain between the Saramacca and Suriname rivers south of Paramaribo.

Field work comprised drilling, geo-electrical and electro-magnetic surveys, pumping tests, water sampling and water level measurements. Furthermore, water samples were analyzed on chemical parameters and O18-, Tritium- and C14-isotopes.

5.3 REGIONAL MATHEMATICAL MODEL

5.3.1 General

On the basis of existing data from amongst others previous studies and data acquired for this study, a regional mathematical model has been developed of the Zanderij and Coesewijne aquifers, the potential sources for the projected expansion.

The purpose of this model is:

- to acquire insight in the regional groundwater flow systems;
- to determine the impact of 4 well-field alternatives;
- to be used as a management tool for further development and management of groundwater in the Zanderij aquifer.

The model simulations led to a better understanding of the groundwater flow systems in the coastal zone of Suriname. During previous studies, no coherent conceptual model had been developed.

5.3.2 Potential well-field alternatives

Four alternative well-fields have been chosen, taking into consideration the pattern of fresh and brackish groundwater, the thickness of the aquifers, the lengths of the well-fields and environmental factors. Three of the alternatives, the Rijsdijk, Zanderij-Vierkinderen and Hanover well-fields, had been the subject of several previous hydrogeological studies. The fourth, Rijsdijk-Lelydorp along the van Hattem road west of Lelydorp village, is a new alternative (see figure 5.1).

The hydrological impacts are determined for a capacity of 2,000 m^3/hr.
For the modelling of the Zanderij aquifer the numerical groundwater flow package TRIWACO (developed by IWACO B.V.) has been used.
The model area amounts to 2,738 km^2 and covers the area between the savannah belt in the south, the Saramacca and Suriname rivers respectively in the west and east and the east and the coastline in the north. The groundwater model has been developed, calibrated and verified using all available information. The groundwater model consists of 3 aquifers:

- phreatic aquifer: Lelydorp sands
- Zanderij aquifer
- Coesewijne aquifer.

An important aspect of the groundwater modelling is the interaction between surface water and groundwater. Due to groundwater recovery, the discharge of surface water will decrease and the recharge of the groundwater will increase (induced recharge).

Potential well-fields Rijsdijk and Rijsdijk-Lelydorp
The hydrogeological conditions at the potential well-fields Rijsdijk and Rijsdijk-Lelydorp are characterized by:

- a relatively high transmissivity of the Zanderij and Coesewijne aquifer;
- a very high hydraulic resistance of the Coropina clay layer.

A groundwater abstraction at these well-fields causes the following hydrological impacts:

- large drawdown of the piezometric level in the aquifer;
- small drawdown of the phreatic level;
- recharge is induced over an extensive area;
- small reduction of the base flow of creeks;
- minor increase of salinization on regional scale.

Because the northern parts of the Zanderij and Coesewijne aquifers are thick, salinization is a relatively slow process.

Potential well-fields Zanderij-Vierkinderen and Republiek-Hanover
The hydrogeological condition at the potential well-fields Zanderij-Vierkinderen and Republiek-Hanover are characterized by:

- a relatively low transmissivity of the Zanderij and Coesewijne aquifer;
- a relatively low hydraulic resistance of the Coropina clay layer.

A groundwater abstraction at these well-fields causes the following hydrological impacts:

- small drawdown of the piezometric level in the aquifer;
- large drawdown of the phreatic level;
- recharge is induced locally;
- large reduction of the base flow of creeks;
- no increase of salinization on regional scale.

The hydrological impacts on the potential well-fields are listed in table 5.1.

Table 5.1: Evaluation of hydrological impact on potential well-fields.

Hydrological Impact	Potential Well-field			
	Zanderij-Vierkinderen	Rijsdijk	Rijsdijk-Lelydorp	Republiek-Hanover
Drawdown phreatic level	***	*	*	***
Impact on other well-fields:				
- Drawdown piezometric level	*	**	***	*
- Salinization	o	*	*	o
Reduction base flow of creeks	***	**	*	*
Expected lifetime	Unlimited	> 30 years	> 50 years	Unlimited

Legend:
o no impact
* minor impact
** moderate impact
*** major impact.

The choice of the well-field cannot be made by only considering the hydrological impacts. Also the environmental impacts and the financial aspects should be taken into account.

With respect to the final selection of the well-field alternative, the hydrological impacts have to be translated into environmental and financial impacts.

5.4 FINAL EVALUATION

A comparative feasibility study was carried out for the 4 alternatives. With the regional model, hydrological and environmental effects were predicted and the financial, technical, hydrological and environmental aspects were evaluated and compared. The Rijsdijk-Lelydorp alternative, clearly distinguished itself as the most favourable option, despite the fact that exploitation at this site (and also at Rijsdijk) is finite because of the slow intrusion of brackish groundwater. Conclusive was the relatively long lifetime of the Rijsdijk-Lelydorp well-field, calculated with the regional model and later with the analytical model AQUISOFT. According to a pessimistic scenario, the chloride concentration would exceed the critical value of 200 mg/l after 50 years.

The WHO-standard for chloride is 250 mg/l. The 200 mg/l concentration is taken as a critical value because water from the new well-field will be mixed with water from the existing SWM well-fields, which have increasingly high salinities (> 250 mg/l). Salinity forecasts have been made for the existing well-fields with the analytical model.

5.5 WELL-FIELD DESIGN

The pumping tests and the observation wells provided the hydraulic parameters for the detailed design of the Rijsdijk-Lelydorp well-field. With the analytical model the optimum location and layout of the well-field was determined. The optimization parameter was the length of the well-field as this is the most cost-determining factor. Constraints were a maximum drawdown of 18 m in the wells and a minimum of 50 years of good quality water. The designed well-field consists of 22 wells with capacities of 100 m3/hr each, aligned over a length of 8,000 m along the van Hattem road, starting 1,000 m from the Pad van Wanica. The wells have an average depth of 70 m and the intermediate spacing between the wells varies between 200 m at ends to 500 m in the centre of the well-field. Two wells serve as standby wells in case of repair and maintenance.

The well-field is divided in two parts, with two independent collector mains, which transport the raw water to the new water treatment plant, located along the van Hattem road, approximately 800 m from the Pad van Wanica.
In the wells submersible pumps (Grundfoss SP70 or comparable) will be placed. The total power consumption of the well-field at maximum capacity amounts to 515 kW.

6 REHABILITATION OF EXISTING WELL-FIELDS

6.1 METHODOLOGY

The aquifer, the wells and the collector pipes form a hydraulic system in which a flow of raw water is activated by means of the submersible pumps in the wells. Boundary conditions of the system are the static water level in the aquifer and the elevation of the sprinklers above the filter beds in the treatment plant, which form the discharge points of the system. Flow of water through this system is accompanied with head losses. The head losses before the submersible pumps consist of aquifer flow losses and well entrance losses. The pump is the place where energy is added to the system by generating the discharge against a pumping head (head gain). After the pump the head decreases again as a result of friction losses in the collector pipes.

The performance of the wells can be evaluated on the basis of two different factors. The first is the specific yield, which is defined as the well discharge divided by the drawdown of the water level in the well. The drawdown combines the aquifer and the well losses. Because the aquifer losses are independent from the well, the well performance can be better evaluated on the basis of the well efficiency, which is defined as the percentage of the aquifer losses to the drawdown.
In Technical Note 5 (see annex 5.8) all existing wells have been evaluated according to specific yield and well efficiency. The basic data have been acquired during the feasibility study of 1987. The recent measurements of December 1990 to March 1991 have been taken into account as well (see annex 6.2). These results have been incorporated in the analysis of the hydraulic system of the various well-fields (see annex 6.1). To determine the specific yield the "free flowing" well discharge was measured after disconnecting each well from the feeder pipe to the collector system. In general existing wells with large well losses (efficiency less than 40 %) should be regenerated or redrilled. For new wells with a proper design as recommended in Technical Note 5 (annex 5.8), an efficiency of 70 % should be a minimum requirement.

The discharge of a pump varies with the pumping head. The pumping head for the submersible pumps is the difference between the dynamic water level in the well and the head in the collector pipe near the well.
A well pump is operating at maximum efficiency with regard to its energy consumption when the actual pumping head is about 2/3 of the maximum theoretical pumping head stated by the manufacturer.
For each well the pumping head was determined on the basis of measurements of dynamic water levels in the well and pressures in the collector pipes. Also the elevation of the points of measurement and the calculated friction losses in the rising pipes and collector pipes were taken into account (see annex 6.1).
The individual well discharge cannot be measured while the wells are connected to the collector system, because there are no water meters. Therefore first the specific yield is determined on the "free flowing" discharge and the corresponding drawdown. Then the specific yield is multiplied by the drawdown in the well, measured while the well is pumping in the system. Although the specific yield is not constant, it is assumed that the variation is relatively small in the range between the "free flowing" discharge and the "in system" discharge. A final correction is applied after comparing the sum of all well discharges to the total discharge measured in the filter room of the treatment plant (see annex 6.1).
Afterwards the well discharge and pumping head are plotted on the theoretical Q(discharge)-H(head) curve of the specific pump as given by the manufacturer.
When pumps operate far outside the area of optimum efficiency, they should be replaced by other pump types. Pumps which do not fit on the curve at all may have been worn and should also be replaced.
If the dynamic water level is close to the pump, the pump should be lowered. If however, the well drawdown is large due to well losses the well should be regenerated or redrilled.
Most of the wells have been equipped with Grundfoss submersible pumps of various types. For all these types theoretical Q-H curves are available. For the two Utah pumps and the four Deming pumps no Q-H curves could be found. The performance of these pumps could not be analyzed according to the method described above.

The collector system comprises all pipes from the submersible pumps in the wells to the sprinklers above the filter beds in the treatment plant.
At various points in the collector systems of the well-fields, pressures have been measured. After correction for elevation of the measurement points, the friction losses have been determined (see annex 6.1). Where relatively large friction losses occur as compared to theoretically calculated values, pipes may have to be cleaned. In other cases large friction losses may be due to the small diameter of the pipes. These pipes may have to be replaced by pipes with larger diameters.

6.2 CONCLUSIONS WK-PLEIN WELL-FIELD

Although all wells are in need of regeneration or replacement, the wells O1A, O2A, S1A, T2C, M2A, C3A, C5A, C9A and E2A are in a particular bad condition and need to be improved as soon as possible according to the specifications described in Technical Note 5 (see annex 5.8).
Well C2A needs to be repaired (pump fallen down).
The pumps in wells S1A and M1A need to be replaced by Grundfoss SP45-4 pumps.
The friction losses over the sprinkler pipes need to be reduced to about 2 m by increasing the number of sprinklers.
The measurements and the analyses have been carried out in December 1990 and January 1991. In the meantime the SWM has already carried out some of these rehabilitation activities: wells C2A and M2A are in production again, whereas some wells have been regenerated. A new well CAT3A has been connected, while T2C will be replaced soon by a new well. In all wells SP45-4 pumps have been placed. Also extra sprinkler pipes have been added.
By mid April 1991 17 wells are in production with a total yield of 660 m^3/hr. Well O1A is temporarily out of production because of pump failure.

6.3 CONCLUSIONS LIVORNO WELL-FIELD

The wells D7A and D9A need to be regenerated or redrilled. The SP35-4 and SP35-5 should be replaced by SP35-7, SP45-5 or SP70-3 pumps.

6.4 CONCLUSIONS LEYSWEG WELL-FIELD

Regeneration of the wells L1C(91), L2C and L5C is recommended. The SP35-5 pump of L1C(91) should be inspected.

6.5 CONCLUSIONS REPUBLIEK WELL-FIELD

Regeneration of wells 31 and 34 is recommended. The wells 7, 10, and 23 should be replaced by two wells between the existing wells 14 and 24 along the southern well string and one well between wells 27 and 30 along the eastern well string. In general wells should be placed further away from each other. The SP27-3 pumps should be inspected (wear and tear). If wells are not redistributed they should be replaced or if possible one of the impellers should be removed.

7 ENVIRONMENTAL IMPACT ASSESSMENT

7.1 INTRODUCTION

Special Report 5: Environmental Impact Assessment, which was issued in November 1990 and chapter 7 of Volume 2 of this Final Report, present the results of an Environmental Impact Assessment (EIA) of the implementation of the Paramaribo Water Supply Project.
The emphasis of this exercise is on the new groundwater abstractions and treatment works in the area south of Paramaribo. In this chapter the results have been summarized.

7.2 GENERAL

The EIA starts with a description of the base-line conditions of the project area environment as far it is likely to be affected. Autonomous developments and trends are also taken into account.
By combining base-line conditions and proposed project activities, potential impacts are described and are further elaborated upon. The potential impacts of the environment (i.e. land use and human activities) on the viability of the project are also considered.

A description of base-line conditions includes aspects such as geographic setting and geomorphology of the project area, data on climate and population and descriptions of water resources, ecosystems, land use and existing sanitary facilities.

It has been concluded that the environmental impacts of the construction phase of the water supply project will be limited and of a temporary nature.

Relevant impacts to be anticipated during the operational phase of the project include:

- impact of groundwater abstractions;
- impact of the discharge of treatment plant wastes;
- impact of an increased waste water flow.

The abovementioned impacts have been further elaborated upon in Special Report 5 and a summary has been provided in the following paragraphs.

7.3 GROUNDWATER ABSTRACTION

For the development of a new well-field for the water supply of Paramaribo and its Metropolitan area, four alternative areas are considered. For each of the well-field alternatives the environmental impacts of drawdown of phreatic groundwater tables have been assessed. Drawdowns have been calculated using the regional, mathematical groundwater model. Based on calculated drawdowns and an assessment of actual ecosystems and landuse it is concluded that the impacts of well-field development in the Rijsdijk-Lelydorp area and in the Rijsdijk area are negligible to limited.

Well-field alternatives in the Zanderij-Vierkinderen area and in the Republiek-Hanover area are likely to create moderate to significant impacts on ecosystems and forms of landuse.

When considering the potential impact of the environment (i.e. human activities and landuse) on the planned groundwater abstraction, it is concluded that the northern well-field alternatives Rijsdijk-Lelydorp and Rijsdijk are less vulnerable to groundwater contamination than the southern well-field alternatives of Zanderij-Vierkinderen and Republiek-Hanover. This is basically due to the presence of a layer of impervious Coropina and Demerara clays which decreases in thickness in a southward direction.

A limited programme of sampling and analysis of both soil and groundwater inside and outside the spoil area, located in the Rijsdijk area, was executed in order to investigate the pollution hazards. As reported in Special Report 5, initially two samples of shallow groundwater in and outside the spoil area contained concentrations of mercury well above the WHO-standards. The high mercury concentrations were caused by the fact that unfiltered samples were taken. Filtered samples, taken at a later date at the same locations did not show any trace of mercury. Also a groundwater sample from the Zanderij aquifer, taken from a nearby well (RL 28/90) was free from pollution.
It is noted that migration of heavy metals to the deeper strata is not likely. These substances will be absorbed by the thick impervious clay layer above the aquifer.

From the environmental points of view, the selection of the northern well-fields is recommended. It is however, concluded that after the ending of mining activities in the bauxite area (anticipated in the year 2005), the mined-out land will be included in capture zones of these northern well-fields. Especially the Lelydorp III mines, still to be developed, will then be located in 25 years zones of the abstractions. Considering the fact that the bauxite belt is located within the Zanderij aquifer, mining activities will create a direct hydraulic contact to the aquifer.

In order to protect the potential well-fields, a differentiated protection strategy is recommended. Levels of protection are differentiated depending on the vulnerability of the aquifer and a system of protection zones.

Establishment of protection zones and a system of prohibitions and permits in order to regulate activities within protection zones should be supported by a framework of legislation and institutional development.
It is therefore recommended to investigate the possibilities and consequences of a groundwater protection strategy in view of regional planning and development, legislation and institutional aspects.

Monitoring activities are recommended, irrespective of the well-field selected. A design study on establishment of the monitoring network including appropriate materials and construction practices is recommended. An appropriate design of a monitoring network at the well-field can be made during the detailed design of the well-field.

If the well-field alternative Rijsdijk-Lelydorp or Rijsdijk will be selected for the water supply objectives, it is strongly recommended to start up discussions with the operator of the mining joint venture (Billion Maatschappij Suriname). Discussions should include the objectives and possibilities of rehabilitation of the mining area in general and the mining operations and future use of the Lelydorp III mines in particular. Future points of conflicts with the water supply objectives should be prevented in an early state.

7.4 TREATMENT PLANT WASTES

Operation of the envisaged water treatment plant will result in a periodical discharge of filter backwash water. It is estimated that on a daily basis approximately 500 kg ironhydroxide will be discharged into the environment. It has been concluded that the iron(hydroxide) itself is not toxic, although the formation of sludge blankets near the point of discharge and increased levels of turbidity may create negative impacts on aquatic life. A major point of concern might be aesthetically undesirable effects due to the reddish-brown colour of the ironhydroxide flocs.

Discharge into creek systems (Tawajari creek) is not recommended, although impacts on aquatic ecosystems are anticipated to be limited.

The discharge of backwash water into the drainage system along the Pad van Wanica would be evident. From an environmental point of view the discharge will hardly create any impact except for an aesthetic impact. It can safely be assumed that aquatic impacts will be non-existent.
The drainage is in northward direction and is intercepted by the Tout Lui Faut canal, a major drainage canal in the area, and finally discharging into the Suriname river.
However, due to its hydraulic gradient, the existing drainage canal or any newly laid canal, along the Van Hattemweg cannot handle a periodical discharge of 1,500 m^3/hr (416 l/s).

Consultants have proposed to construct an equalizing/sedimentation basin, close to the site of the new WTP. By settling, 50-70% of the suspended matter can be removed from backwash water of ground water treatment plant with detention times of 4-24 hours and an effective settling depth of 1.5 metres. Applying anionic poly-electrolyte dosage up to 1 mg/l, removal efficiencies can be increased up to 85-90% (KIWA, 1981).

The basin has been sized in such a way that a sludge storage for approximately half a year can be achieved.
The supernatant water will have to be discharged on the close by swamp with a continuous flow rate of approximately 160 m^3/hr. The discharge rate of water into the drainage system will thus be reduced by a factor 9-10.

7.5 INCREASED WASTE WATER FLOWS

Depending on the applied economic growth scenarios, total water consumption in the year 2000 will increase with 20 to 40% as compared to the present situation. For the year 2010 this figure will be 40 to 70%. When it is assumed that the water consumed is converted into waste water, the above given figures give an indication of the overall increase in waste water discharge in the SWM service area.

The present sanitary system in the Paramaribo area is characterized as a combined sewerage and drainage system receiving the following components:

- sanitary and domestic waste waters;
- industrial and institutional waste water;
- stormwater runoff.

The hydraulic capacity of this system basically has been designed in order to deal with the stormwater runoff. It can be concluded that the total waste water flow is negligible as compared to the amounts of stormwater flow to be drained. Hence, the impact of (additional) waste water flows on the existing system can be considered insignificant. No changes in design criteria or capacity of the system merely as a result of increase waste water flows are required.

However, in the dry season, when waste waters mainly compose the water flow through the sanitary system, an increase in waste water volumes undoubtedly will create additional unhygienic conditions already being present.
Increasing waste water flows, finally discharged without additional treatment into the Suriname river also will further deteriorate the water quality of this river. Impacts of present waste water discharges are already noticeable, especially at the point of discharge of the Saramacca Canal where industry is concentrated.

Outside the urban areas of Paramaribo and in the SWM supply area in the Wanica and Para Districts, increased waste water flows will increase risks of shallow groundwater contamination due to absence of adequate drainage systems and the application of pit latrines for excreta disposal. Since in these areas the population also partly relies on non-piped water sources, public health hazards also are likely to increase.

The improvement and expansion of the Paramaribo water supply system aims to increase the provision of safe potable water in sufficient quantities. Hence the implementation of this project consequently aims to improve standards of living, increase well-being and increase public health standards.

In order to enhance to abovementioned objectives it is strongly recommended to improve existing sanitary conditions.

8 LABORATORY

In Special Report 6: Laboratory, issued in November 1990 and in chapter 8 of Volume 2 of this Final Report, a description is given of the existing SWM laboratory and its activities. Based on a set of proposed monitoring activities, recommendations are made on the improvement and expansion of the existing laboratory. A summary of the conclusions and recommendations has been provided in this chapter.

Present monitoring activities as executed by the SWM laboratory include the sampling and analysis of production wells and process units of existing production facilities. The analyses carried out comprise a relatively limited set of parameters.

Recommendations are made on a more comprehensive system of monitoring activities. Distinction is made between the following monitoring activities:

- monitoring groundwater resources, including production wells and observation wells (to be installed);
- monitoring treatment plant performance;
- monitoring produced water and water quality in the distribution system.

For each of the above mentioned activities a different set of parameters to be analyzed and a monitoring frequency is proposed.

The proposed monitoring activities will increase the work load of the existing laboratory. It is concluded that the laboratory needs to be expanded and upgraded.

Recommendations on the improvement of the existing laboratory dealing with equipment requirements, required laboratory space and required staffing and training. Considering the amount of analyses to be carried out, it is also recommended to implement a computerized database system for data-handling and reporting purposes.

9 CLEAR WATER TRANSMISSION MAINS

In Special Report 11: Clear Water Transmission Mains, issued in January 1991 and in chapter 9 of Volume 2 of this Final Report, the required transmission mains have been dealt with. A summary has been provided in this chapter.

The new water supply system for Paramaribo requires the construction of two clear water transmission mains:

- a main pipeline from the WTP at Lelydorp to WK-plein at the centre of the city and

- a connecting transmission main to transport water from WK-plein to Blauwgrond, located in the north of the city.

Several pipeline materials were evaluated for their suitability to transport the envisaged quantities. Steel mains (cement lined, PE coated and with cathodic protection) were selected as the most suitable for the required sizes and design duties.

The capacity requirements for the transmission main from the WTP to WK-plein have been established from the socio-economic studies of future demand and the envisaged production capacity to meet the anticipated needs. Three economic growth rates were postulated culminating in a similar number of demand growth scenarios.

The total cost of a pipeline comprising capital investment costs and recurrent operating (energy) costs was determined for a range of pipeline nominal diameters (400 - 1,000 mm). The cash flows associated with each pipeline were calculated starting in 1993 (the envisaged date for construction, and based on the estimated demands using the "high" growth scenario in Phase I only and Phases I and II combined. An economic assessment based on Phases I and II up to the year 2013 was considered to be more appropriate to a pipeline project where an economic lifetime of over 20 years is usually applicable. The cash flows have been discounted at a rate of 12% to determine the Net Present Value NPV for each pipeline diameter. The optimum diameter, from an economic point of view, meeting the high growth rate requirements for Phases I and II, was found to be a 800 mm nominal diameter pipeline.

The economic sensitivity of the water transmission main projected for Phase I, to changes in demand from the base case of the "high growth scenario" was assessed by considering the NPV's associated with "medium" and "low" growth in demand. Similarly, the sensitivity of the transmission main projected for Phases I and II combined was assessed by estimating the NPV's associated with the "high growth rate" base case and the "medium" and "low" scenario.

When testing the sensitivity of the economic diameter to variations in capital cost it was found that the selected diameter would be the same in all analyzed cases.

The sensitivity to increases in energy costs of the selected pipeline was tested at the three growth scenario of "high" (the base case), "medium" and "low" and found to be highly significant.

The above analyses indicate that the 800 mm transmission pipeline is the most economic diameter, offering the least risk to changes in the principle cost parameters.

Budgetary type cost estimates of the transmission main from the WTP to WK-plein as well as of the transmission main from WK-plein to Blauwgrond have been prepared at the conceptual design stage and presented at the end of this report.

Recommended measures for future expansions to meet the demand beyond the design period are proposed.

10 DESIGN DISTRIBUTION SYSTEM

Special Report 7: Design Distribution System, issued in December 1990 and in chapter 10 of Volume 2 of this Final Report, a thorough review of the existing distribution systems in the SWM service area has been given and recommendations have been made with regard to the improvements that are required in order to provide at adequate pressure the future water demands in the service area. In this chapter a summary of the findings and recommendations has been given.

The designs are based on the high growth scenario of the projected consumption rates within the distribution area for the year 2000.
The existing distribution system is divided into 3 sub-systems:

1. Paramaribo Distribution System.
2. Republiek-Main Distribution System.
3. Livorno Distribution System.

The total amount of water to be distributed in the present service area will increase from 811 l/s in 1989 to 898 l/s in 2000. At the same time, the net unaccounted-for water is projected to decrease from 25.5% to 20%.
The existing distribution systems will be extended and modified in order to supply at adequate pressure the increased amounts of water. Additional storage capacity will be provided and approximately 52 km of primary distribution mains will be laid.

The future primary distribution systems will consist of two systems, namely the Paramaribo Distribution System and the Republiek-Main Distribution System. The primary distribution system is defined as the network consisting of major distribution mains with pipe sizes larger than 200 mm. The Livorno Distribution System is considered as a reticulation system, and the design of the required extensions will be based on the design criteria, developed in the "Design Manual for Reticulation Systems".

A computer model of each distribution system has been prepared and various pipe network computer analyses of the proposed distribution system have been performed, using projected year 2000 water demands. The results of these computer analyses provide the necessary information for developing the most cost-effective combination of facilities required to provide reliable water service to the consumers in the service area.

The total length of new primary distribution mains amount to 52 km, which is divided over the two distribution systems as presented in table 19.1.

Table 10.1: Additional lengths in m of pipeline in the primary distribution system in 2000.

Distribution Area	Diameter					
	200	300	400	500	600	Total
Paramaribo Distribution System	10,650	5,850	9,900	3,000	1,555	30,955
Republiek-Main Distribution System	1,200	8,550	-	11,200	-	20,950
Total	11,850	14,400	9,900	14,200	1,555	51,905

The pipe lengths is table 10.1 are derived from the hydraulic calculations.
A new reservoir will be built at WK-plein, which will provide an additional storage capacity of 8,000 m³. The new reservoir will be connected to the Blauwgrond reservoir through a 500 mm transmission pipeline. The required storage capacity of 3,900 m³ at Lelydorp will be integrated in the clear water reservoir of the new treatment plant.

A set of design criteria has been prepared including pipe materials, valves, pressures in the distribution system and Fire-Fighting requirements. The designs of the primary distribution mains are based on PVC for pipes with a diameters between 200 mm and 400 mm, and steel for pipes larger than 400 mm. A summary of the selected materials for design is presented in table 10.2.

Table 10.2: Pipe materials for the primary distribution networks

Nominal diameters (mm)	Proposed materials (1)	Materials assumed for design purpose	Internal diameters for design purpose (mm)
150	PVC	PVC	144
200	PVC	PVC	180
300	DCI, ST, PVC	PVC	250, 280
400	DCI, ST, PVC	PVC	386, 380
> 400	DCI, ST	ST	483, 584

(1) The abbreviations stand for the following pipe materials:

DCI = Ductile cast iron.
PVC = Polyvinyl chloride.
ST = Steel.

11 RETICULATION SYSTEM DESIGN CRITERIA

The reticulation system is defined as the network of small diameter pipes (laterals) which are tied to the primary distribution system, and on which the service lines are connected. The diameter of reticulation pipes ranges from 63 mm (ND) to 160 mm (ND).

In Special Report 15: Reticulation System Design Manual, issued in February 1991 and in chapter 11 of Volume 2 of this Final Report a method has been presented with which a balanced reticulation system can be designed.
This method, or reticulation system design manual, is intended for the (reticulation system) design team of the SWM.

Backgrounds, used for the design of smaller distribution systems have been provided, as well as descriptions on prevailing consumption rates, required pressures, pipe materials, relevant fittings, appurtenances, and peak factors.
Design tables have been developed, which provide information on the number of houses to be served, the applied peak factors, the resulting flow through the selected pipe, the water velocity and head loss.

12 SURGE ANALYSES

12.1 INTRODUCTION

In chapter 12 of Volume 2 of this Final Report deals with the analyses of water hammer in the water transmission mains designed or under review. Protective measurements may be needed to protect the water transport installation against damage, caused by water hammer for example in the event of pump failure of all pumps. Water hammer leads to unwanted under-pressure c.q. cavitation which in its turn leads to uncontrolled pressure peaks.

The uncontrolled peaks in the pressure arise when the cavitated area is phased out. This can lead to damage to the mains and pumps. Mains may get bursted, or be placed out of position. For steel pipes with cement lining, the lining might be seriously damaged.

At times of pump failure, no under-pressure is therefore allowed to occur whereas the maximum pressure shall not exceed the pressure in the steady state condition. A computer program has been used to simulate possible pump failures. All calculations have been repeated several times to find the optimal solution.

For the Paramaribo water supply system, five water supply mains have been analyzed. The situation without protection has been analyzed to assess the need for such a protection and the situation with a protection has been analyzed to know the behaviour of the system. The analyses of the systems are based on the worst case circumstances.

Conditions are:

For PVC pipes : no under-pressure at all (to avoid leakage trough connections);
For steel pipes : no pressure below 3 mwc (no cavitation allowed).

As protective measurements stand-pipes, air vessels or air valves can be placed or the momentum of inertia of the pumps can be increased. Under all these options the effects of the velocity change are limited. However, in the case of Paramaribo stand-pipes are no realistic option considering the required head in the pipes. The momentum of inertia is fixed by the choice of pump and is not always really free to choose.

The results of the surge analysEs indicate that cavitation occurs due to pump failure on several transmission mains. Under these conditions there is no safety against water hammer. Air vessels are required on the transmission mains from the WTP to WK-plein (3 * 50 m^3), from WK-plein to Blauwgrond (45 m^3) and from Republiek to Lelydorp (1 * 20 m^3 and 1 * 40 m^3). The transmission main from Livorno to WK-plein will be protected against water hammer by increasing the momentum of inertia of the new pumps. In the well-field the water hammer protection will be provided by triple function air valves, to be installed at each well-head and at three locations of the raw water header.

13. UNACCOUNTED-FOR WATER

13.1 GENERAL

In Special Report 8: Unaccounted-for Water, issued in January 1991 and in chapter 13 of Volume 2 of this Final Report, an analysis has been given concerning the prevailing unaccounted-for water. First, an estimate has been made with regard to the actual production and distribution figures. Secondly an evaluation of the metering practices has been made. Finally recommendations have been prepared for a leakage control pilot programme in a representative area of Paramaribo and for a water distribution management system. The recommended leakage control pilot programme has been implemented and is discussed in chapter 13 of Volume 2 of this Final Report. In this chapter a summary of the results and recommendations has been presented.

The corrected production figures of the water production stations over the year 1989 are:

Republiek	3,092,000	m^3/year
WK-plein	5,429,000	m^3/year
Leysweg	3,672,000	m^3/year
Livorno	3,780,000	m^3/year
Lelydorp	2,156,500	m^3/year
Flora	244,500	m^3/year
Total	18,374,000	m^3/year

The authorized/claimed water use has been calculated to be:

Registered consumption	13,650,425	m^3/year
Under-registration water meters	273,000	m^3/year
Tank-car	520	m^3/year
Fire fighting	6,825	m^3/year
Fire hydrant maintenance, use	3,800	m^3/year
Installation, repairs	1,000	m^3/year
Flushing	5,000	m^3/year
Reclaimed illegal used water	7,500	m^3/year
Total	13,948,070	m^3/year
The quantity of water paid:	18,662,500	m^3

?

The unaccounted-for water percentage amounts to 24%. However, the quantity paid is 101.6% (both calculated as percentage of the net production).

The newly arrived Signet flow meters for use at the main water production centre do not function satisfactorily yet. Absence of permanent metering makes the preparation of a water balance less accurate. Provision of reliable metering of the quantity of water produced is strongly recommended.

Presently part of the distribution system still consists of 9.5 km of steel pipes laid in 1934. These shall be replaced or at least taken out of use.
Incrustation on large scale is reported to occur in cast-iron special (bends, tees, valves) used in the PVC and AC piped systems.

A programme for maintenance or replacement has to be carried out starting from the high pressure sides in the system c.q. in circles around the pumping stations.

Many activities of the customer complaints and repair section concern the house installation and thus are not essential for the Water Supply Company and are recommended to be left to contractors.

Water pits for fire fighting are recommended to be replaced by fire hydrants. For the town centre and/or shopping areas preference shall be given to above ground hydrant.

It is recommended to introduce facilities for filling tank-cars with drinking water.

A more intensive reporting system on leaks and subsequent repairs should be introduced. This will ease the stock keeping and decision making about replacement or abandoning of pipelines or parts thereof.

Quality control should be exercised not only on materials arriving in the SWM stores but also materials in stock.

It is recommended to increase the number of field controls on premises which either have or have not been connected before, in order to establish the need of a new connection or the presence of an illegal connection.

Cross control of the data base of location of water meters and the customer data base is recommended to search for lost meters or lost customers.

Additional staff and equipment shall be arranged for the execution of testing and field control of water meters in order to assure that the maintenance and calibration schedules are executed properly.

The water meter workshop should be fully equipped for washing, painting, degreasing and testing of all water meters of SWM and DWV. It is noted that due cooperation will be required between the SWM and DWV in the framework of purchasing new, standardized water meters. This in order to prevent difficulties with spare parts.

In view of the accuracy of volumetric type water meters it should seriously be considered to repair and re-use this type of meters (make BR).

Meters should be installed in such a way that these are accessible also during the rainy season. Simultaneously exposure to direct sunlight will have to be avoided as this has a bad influence on the quality of the water meter especially during periods that no water is drawn from the connection.

It is recommended to improve the accuracy of registering the actual consumption including a tighter check on the functioning of water meters, and switching over to a house-to-house system of meter reading and checking on the actual water consumption.

The timely replacement and re-calibration of the water meters is an important step to take. It is recommended that water meters are called back for re-calibration within a period of 5 years at the most. Emphasis should thereby be put on the meters of the customers with the highest consumption rates, especially those among the industrial and large commercial customers.

It is recommended to carry out an integrated leakage control programme consisting of the setting up of consumption districts, and carrying out runs on smaller areas followed by sounding for leaks in suspected areas indicated by these tests and the subsequent repair of these leaks.

The system should be kept under pressure 24 hours per day while preventing excessive pressures in the distribution systems as these will lead to a considerable increase in wastage, but also preventing too low pressures which may cause under-registration of water meters and surging of contaminated water or air into the piped system.

It is recommended to use corrosion-free materials (in- and outside coated/lined steel, or DCI for diameters of 600 mm and more; PVC or AC for diameters of 200 to 500 mm; PVC for diameters of 20 to 200 mm, and PVC or PE for diameters below 32 mm), of a sufficient pressure class, and laid at sufficient depth (at least 0.8 m cover) and where necessary in mantle pipes.

All available data on the distribution and reticulation system should be integrated into one set of accurate and reliable drawings, complemented by bills of quantities for the various pipe diameters, materials, age, valves, etc. Regular field checks will be indispensable to keep the recorded information up to date.

13.2 REDUCTION OF UNACCOUNTED-FOR WATER IN PILOT AREAS

13.2.1 CONCLUSIONS

An Unaccounted-For Water (UFW) reduction programme was set up and introduced on pilot scale in two representative areas of Paramaribo. The conclusions and recommendations of this pilot programme are listed here below.

The results of the improvements for the pilot areas are summarized in table 13.1.

Table 13.1: Summary reduction UFW

	AREA 1	AREA 2	TOTAL
UFW 0	9.4%	25.5%	17.8%
Reduction of UFW by:			
1. Updating administration	1.0%	2.4%	1.8%
2. Replacing defective water meters	0.0%	10.5%	5.3%
3. Leak detection and repair	-1.2%	6.2%	2.7%
UFW after improvements	9.6%	6.4%	8.0%

<u>Present unaccounted-for water (UFW 0)</u>

The present unaccounted-for (UFW 0) is 10% in area 1 and 26% in area 2. In area 1 it is lower than was expected, based on the overall data for Paramaribo. This might be due to the low pressure in this area. Also, the unaccounted-for water for the whole distribution system of the SWM should be calculated more accurately by measuring the flow at the distribution pumping stations.

<u>Improvement 1: updating administration</u>

Updating the administration by cross checking the administrative records and a house to house survey has given an average improvement of 1.8 %. This is a rather low result for the amount of work involved. Especially the cross checking of the administrative records is laborious, and the result only minimal: an average of 0.8% of the customers is administratively lost.
An average of 1.1% of the connections were illegal, three disconnected former customers and 1 registered customer.

The benefit/cost estimates confirm that the effects of this step are not very promising: it is only profitable when the water prices are increased; if not, the benefit/cost ratio is only 1.3.

<u>Improvement 2: replacing defective meters</u>

In the framework of the pilot project, 110 water meters have been replaced because they were unreadable or did not work. The number was reasonably evenly divided over both areas.

The overall results of the replacement of defective water meters are promising, i.e. a reduction of the UFW with 5.3%. However, there is a large difference between the two areas. In area 2 the reduction of the UFW and the profitability are very good. In area 1, however, the improvement is nil. Apparently the estimates used by the administration are rather adequate for areas with low pressure (like area 1). But in area 2, with a higher pressure, the consumption per connection is higher. For area 1, replacement of defective meters may become opportune after the increase of the system pressure.

<u>Improvement 3: leak detection and repair</u>

In both areas, the majority of the leakages encountered occurred at the water meter: about 10% of all connections were leaking. There was hardly any leakage in the distribution system itself. The

reduction of the UFW shows the same pattern as for improvement 2: for area 1, the result is nil, whereas the result for area 2 is satisfactory and profitable.
The overall figures show a reduction of the UFW of about 2%, and a benefit/cost ratio between 2 and 5.

Overall effect of improvements

By the implementation of the improvements the unaccounted-for water was reduced from 17.8% to 8.0% (9.6% in area 1, 6.4% in area 2), which are quite acceptable levels.

The effect in area 1 is nil; the differences are within the margin of error. For area 2 however, the improvement is significant and promising.
The main conclusion of the pilot project is that the reduction of unaccounted-for water is technically possible and profitable, in areas with high pressure. In areas with low pressure, the effort appears to be not worth while.
However, it is important to realize that after the expansion of the production and distribution capacity, the pressure in these areas will increase as well. Therefore, the leakage will increase and leakage detection and repair will become profitable in these areas.

Main causes for unaccounted-for water

It can be concluded that the main cause for the unaccounted-for water is a large number of defective meters, and for a smaller part illegal or administratively lost connections. Only a very small part of the unaccounted-for water is physical leakage: most of the UFW is not going to waste, but consumed without being paid for.

This means that by the implementation of a project to reduce the UFW the income of the SWM will increase.

However, the level of service to the customers will not be improved: there will not be more water available to the customers, nor will the pressure be higher in the area.

13.2.2 RECOMMENDATIONS

Based on the results of this study for the reduction of the unaccounted-for water in two pilot areas, the following recommendations are made:

Determination UFW for whole Paramaribo

To be able to determine the UFW for the whole distribution system more accurately, the flow should be measured and registered at all production and distribution stations. The implementation of this recommendation is already taking place by the installation of flow meters.

Reduction of unaccounted-for water

After the pilot projects, a scheme should be set up for the implementation of a programme to reduce the unaccounted-for water in Paramaribo on a larger scale. Based on the results of this study, the following suggestions can be made:

- The implementation of the whole programme will take a number of years. The programme should first concentrate on the areas with higher pressure. The areas with low pressure have a lower priority;
- The results of the programme should be evaluated regularly by determining the UFW in small areas before and after the improvements. If necessary, the programme should be adjusted;
- The programme for reduction of the unaccounted-for water should concentrate on the replacement of old defective meters, as this is the most profitable step;
- Parallel to this, the water meter workshop should be upgraded, and a programme should be set up for the regular maintenance of all water meters (once every 5 years);
- A regular house to house survey is useful, especially to find illegal connections. It should be executed once every 5 years. However, it is very laborious. If the required manpower to execute a survey regularly is not available, at least the disconnected customers should be checked regularly, as these are the most probable illegal connections;
- The administrative records should be updated more consequently, to avoid discrepancies;
- Leak detection and repair should concentrate on the state of the house connections, especially at the water meter. All new connections should be properly executed.

14. PUMP SELECTIONS, POWER SUPPLY AND CONTROL SYSTEMS

14.1 INTRODUCTION

Special report 12: Pump Selections, Power Supply and Control Systems and chapter 14 of Volume 2 of this Final Report, deal with the main electro-mechanical works. Subsequently, the pump selections, the power supply and distribution and the control systems are discussed. In this chapter a summary has been provided.

14.2 PUMP SELECTIONS

For the selection of the required pumps, the system characteristics have been used, that resulted from the designs of the transmission mains, (see Special Report 11 and chapter 9 of Volume 2) and the distribution systems (see Special Report 7 and chapter 10 of Volume 2).

The pump selection has been based on Vertical Industrial Turbine Pumps for all reservoirs and Horizontal Single Stage Double Suction Pumps for the pipeline booster station.
All Q-H curves and pump data has been taken from the latest manual of GOULDS PUMPS, Inc.

These pumps are available with a wide range of capacities. Each type and model can be fitted with at least three different impeller diameters, which allows for an accurate selection. Where possible, pump selection has been based on medium impeller diameters.
Each pump station will be provided with one extra pump as a spare unit. This pump will be of the largest selected type as applicable to the respective pump station.

It must be borne in mind that the information of the above mentioned manufacturer has been used for pump selection purposes only and that this report does not imply that other pump manufacturers would not be equally compatible.

An attempt has been made to standardize as much as possible in order to facilitate the supply of spare parts and to encourage maintenance.

Table 14.1: Selected pumps

Station	Pump Type	No. of Stages	No. of Pumps	KW per Pump	Total KW Required
LELYDORP Distribution	Vertical 14 JLO Vertical 16 DMC	2 2	2 3	32 120	210
LELYDORP Transmission	Vertical 24 CLC	1	3	128	245
WK-PLEIN Distribution	Vertical 18 DX Vertical 24 CLC	1 1	1 4	100 165	435
WK-PLEIN Transmission	Vertical 18 DMC	1	2	115	115
BLAUWGROND Distribution	Vertical 14 JLO Vertical 16 DMC	1 1	2 3	27 95	185
REPUBLIEK Booster Station	Horizontal 6x8-22	1	2	140	140
REPUBLIEK Distribution/Transmission	Vertical 16 BHC	2	2	100	100
LIVORNO Distribution/Transmission	Vertical 12 JLO	2	3	17	50

14.3 POWER SUPPLY, DISTRIBUTION AND CONTROL SYSTEMS

The power supply, distribution and control systems concerning the following sites are covered in this Special Report:

- New Sites
  - Rijsdijk-Lelydorp
  - Republiek Booster Station
- Existing Sites
  - Republiek
  - Livorno
  - Blauwgrond
  - WK-Plein

For the new sites the power requirements have been estimated, based on installed loads and the *expected highest simultaneously occurring loads.*

The existing sites have been visited during a preliminary survey to establish the presently installed loads and the visual state of the equipment. Rehabilitation is envisaged for the mentioned existing sites. Considering the age of the installations, it is recommended to replace most of the existing equipment with new equipment, except at Livorno. For the various sites, installed loads and highest simultaneous loads have been estimated, based on equipment, process and demand requirements.

The most significant data for the electrical power supply system are summarized in table 14.2. The diesel driven electricity generating sets have been selected based on power requirements. When criteria of maintenance, spare parts supply, inter-changeability, etc. are considered, it is found to be an advantage to select three types of generating sets as shown in table 14.2.

Table 14.2: Power Supply Summary

		RIJSDIJK-LELYDORP	REPUBLIEK	WK-PLEIN	LIVORNO	BLAUWGROND
Highest Simultaneous						
Load Estimated	KVA	1,488	468	1,222	388	244
Rounded to	KVA	1,500	500	1,250	400	250
Highest Load to Start						
		Transp. pump Red. torque	Distr/Trans Pump red. torque	Distr. pump reduced torque	Compressor DOL	Distr. pump reduced torque
Type of start	KW	150	115	190	22	110
Starting load	KVA	1,900	755	1,715	465	530
Peak load without filter cleaning	KVA	1,345	350	1,040	360	250*
"Night" load without filter cleaning	KVA	1,115	345	565	270	100*
Calculated diesel gen. required (electric)	KVA	1,520	650	1,375	400	425
Selected diesel gen. - electric	KVA each	2 + 1 800	2 + 1 350	2 + 1 700	1 400	1 + 1 425
Final selection diesel gen.	KVA	800	425	700	425	425

* Filter back washing not applicable.

15 INSTITUTIONAL ASPECTS

In Special Report 14: Institutional Aspects, issued in February 1991 and in chapter 15 of Volume 2 of this Final Report, an assessment has been made of the organizational performance, strengths and weaknesses of the Surinaamsche Waterleiding Maatschappij (SWM) as well as recommendations regarding both on-going activities and the foreseen extensions. A summary of the findings and the recommendations has been presented in this chapter.

Most of the problems faced already for years by the SWM, are stemming from the dominating deteriorating social, economical and political situation in Suriname. Lack of certain key staff, lack of motivation amongst staff and lack of materials, together with a common feeling of a dead-end future are manifest within the SWM. On-going technical and non-technical operations show a satisfying level of performance, but are also clearly affected by the 'major illness of Suriname'.
The inability to fill quite a number of vacancies at higher and middle specialist and management levels, is the major problem for the SWM. As a result of this gap, during the latter years the SWM's managerial abilities, necessary to sustain and increase momentum of the organization, became gradually weaker.

It is obvious that major improvements can only be achieved when the social, economical and political climate changes for a better one. The room for the SWM itself to do much, is therefore very limited.
Despite these limitations, the SWM is recommended to continue and intensify the social renewal of the organization. Ideas and directions on top level are basically right. However, implementation and acceptance is often not yet up to expectations and/or effective. Therefore, external assistance regarding social and organizational matters should be continued and enhanced. At the same time the internal 'striking power', especially of the personnel department, should be upgraded.
SWM's buying power (especially salaries) should be reviewed in order to let the SWM have a better position on the labour market. Unavoidably, water tariffs will most probably increase because of such higher labour costs.

A clearly organizationally separated department or directorate for 'strategy and steering' is recommended to be set up, to give more active shape to the future and to act as the organizational basis for the IADB-extension project.

16. IMPLEMENTATION SCHEDULE

The time required for the implementation of the envisaged works, is a determining factor in the financial analysis of the project. Furthermore, it is important that all parties concerned realize that a considerable time will pass before the new installations will become operational and before the project as a whole will have been finalized. In figure 16.1 the implementation schedule has been presented.

The schedule starts in August 1990, with the beginning of the present detailed design phase of the project. In the second half of 1991, the detailed designs will have been finished and consequently the investment costs to be expected will be known with an acceptable degree of accuracy. Upon completion of the detailed design phase, discussions can be held with regard to the financing of the implementation of the project. It has been assumed that such negotiations will result in a financial agreement early 1992.

As soon as the financial agreement is in force, international tenders can be held for the supply and construction of all new and rehabilitation works. It is expected that the time, required for tendering including evaluation and clarification can be limited to 6 months.

In the second half of 1992, contracts are expected to be signed for the envisaged works. The production of electro-mechanical parts, which mostly require reasonably long production periods, can then be started. It has been assumed that after a period of 6 months, most of the equipment and materials can be shipped to Paramaribo. After this shipment and custom clearance, the equipment and materials are expected to be available in the second quarter of 1993.

All civil construction works, including those for the existing treatment facilities, are expected to be started after a short mobilization period of 2 months. For the existing treatment facilities, temporary units will be installed, which will take over the treatment process during the reconstruction works.

Depending on the actual progress of the civil construction works at the various sites, the installation of electro-mechanical equipment can be started.

Since the civil works for the new reservoirs at WK-plein and Livorno, the pump pit at Blauwgrond and the booster station at Republiek are less complex than the civil works of the new well-field and the new Water Treatment Plant, installation of the electro-mechanical works is expected to start in second or third quarter of 1993. This work however, does not make part of the critical path of the project and considerable slack time is available.

Considering the workload as far as the drilling activities are concerned and taking into account the influence of the rainy seasons on the progress of these specific works and that this activity does not constitute the critical path of the project, it has been assumed that these drilling activities will last for two years. After approximately one year, when part of the wells will have been drilled and the electro-mechanical equipment and materials will be available in Paramaribo, installation of the electro-mechanical equipment and the collector mains can be started.
At the same time, laying of the two transmission mains and of the distribution pipes can commence.

The civil construction works of the new WTP are part of the critical path of the project. It has been assumed that one and a half year after the start of these works, by the end of April 1994, the progress will be such that installation of the electro-mechanical works can be started.

Testing and commissioning of the various individual parts as well as of the integrated system of Well-field, Water Treatment Plant, Clear Water Transmission & Distribution Pumps and Mains and the overall control system are expected to take place in the last quarter of 1994.

Upon commissioning of the new and rehabilitated existing works, the temporary facilities at Flora, Benie and Lelydorp will be taken out of use, as well as the production unit at Leysweg. With the closing down of these facilities, Phase I of the project will then have been completed.

17 FINANCIAL, ECONOMIC AND SOCIAL ANALYSES

17.1 INTRODUCTION

Special Report 14: Financial, Economic and Social Analyses, issued in January 1991 and in chapter 17 of Volume 2 of this Final Report, deal with the financial, economic and social sides of the envisaged project. A summary of the results has been given in this chapter.

17.2 GENERAL

As the first step of the analyses, an evaluation of the performance of the SWM has been made by performing a historical financial analysis, covering the last five years.

Secondly, financial analyses have been made, applying the data of the above mentioned historical financial analysis, the two scenarios concerning the demand projections as presented in Special Report 1 (see chapter 2 of Volume 2 of this Final Report), and the cost estimates which were prepared for the envisaged project. The financial analyses have been made in constant 1989 US$ prices (SPMOD-model of the IADB) and in current Sfl prices (FPM-model of IWACO).

Third, the existing tariff structure has been evaluated and a new tariff structure has been proposed.

Fourth, the envisaged project has been evaluated for economic feasibility by using the SIMOP model of the IADB.

Finally, the social feasibility of the project has been determined.

17.3 COST ESTIMATE

Based on the preliminary designs of the above mentioned main components, a cost estimate has been prepared. In table 17.1 the estimated costs for the various project components have been presented.

The total amount of US$ 69,691,816 is considerably higher than the estimated costs of the Rijsdijk alternative as presented in the Addendum of the Feasibility Study. Several factors can be mentioned for this difference.

Before an elaboration is given of this difference it is important to realize that a slightly different interpretation has been used for the term "rehabilitation works" in both studies. Rehabilitation works under the present study are considered those works that will not result in an increased capacity of whatever facility, but will result in an improved quality of that facility and will thus guarantee the continuity of the system. This means for instance that the construction of a new reservoir at WK-plein will under the present study no longer fall under the category rehabilitation.

First of all, the additionally required production capacity is now 2,000 m^3/hr, against a previous estimate of 1,700 m^3/hr. This is partly caused by the fact that a number of existing, -sometimes temporary- production facilities will have to be taken out of commission due to their physical condition. The increased additionally required production capacity has its effect on the Well-field, the Water Treatment Plant and the Clear Water Transmission Main.

Due to the fact that in the proposed Well-field, wells with considerably higher yield can be made compared to the previously proposed alternative, the number of wells has been reduced drastically from 80 to 22. Furthermore the water will be pumped from the wells directly into the Water Treatment Plant, instead of using an intermediate raw water storage tank and auxiliary raw water pumps. This has resulted in a lower total amount for investment costs of US$ -1,025,000.

The integration of existing distribution facilities in the design of the new Water Treatment Plant, as well as the necessity for the SWM to generate its own power supply for the new WTP and Well-field, -due to the fact that the EBS cannot guarantee the supply of the required power-, are further major reasons for the increase in investment costs of the new Water Treatment Plant with US$ 3,551,000.

The required increased transport capacity of the Clear Water Transmission main from the new WTP to the Distribution Centre at WK-plein has resulted in an optimum diameter of 800 mm against the formerly assumed 600 mm. This has caused a drastic increase in investment costs of US$ 4,676,000.

The investment costs for the construction of the new Distribution Centre at WK-plein are considerably higher (US$ 5,913,000) compared to the estimated investment costs of the Feasibility Study, mainly due to the fact that besides a larger new Clear Water Reservoir, also Transport pumps for the Blauwgrond reservoir, Distribution pumps and Power Generating Facilities are required.

In the Feasibility Study, no provisions were included for the laying of the Clear Water Transmission main from WK-plein to the Blauwgrond reservoir (US$ 2,127,000), nor for the replacement of pumps at Blauwgrond, Republiek and Livorno (US$ 6,395,000) and neither for the extension of the Distribution system (US$ 12,468,867). Since the rehabilitation of the distribution system is difficult to separate from the new works and can only be considered integrally, the latter amount also includes the cost of necessary rehabilitation. Furthermore the cost for the construction and fitting up of a Water Meter Repair Workshop, which is desperately needed, has now been included.

The rehabilitation needs of all the existing treatment facilities have been re-evaluated. It appeared that besides replacement of the previously defined electro-mechanical works, all production facilities are in need of a drastic rehabilitation of the civil structures as well.

The rehabilitation works for the WK-plein, as defined under the Feasibility Study, also comprised the construction of a new reservoir. In the present set-up this has been included in the new works (Distribution centre WK-plein).

Since the Leysweg and Lelydorp facilities will be taken out of production, rehabilitation has become unnecessary.

Considering the fact that the SWM recently received a new drilling rig from another source, this item has now been deleted from the list.

The total difference in investment costs between the Feasibility Study and the present Detailed Design Study, as far as the rehabilitation works are concerned, amounts to US$ 1,791,000.

Table 17.1: Cost estimate in US$ (price level end 1990)

Production capacity: 2,000 m3/hr			Investment Costs		
Nr	Description	FOREIGN COMPONENT	Local Component		Totals
			Skilled	Unskilled	
1	Well-fields	2,892,633	1,123,386	1,261,219	5,277,239
2	New Water Treatment Plant	6,168,201	2,612,652	2,131,152	10,912,005
3	*Clear Water Transmission (WTP-WK-Plein)*	5,966,713	698,622	1,624,116	8,289,450
4	Distribution Centre WK-Plein	4,366,673	1,704,499	1,068,649	7,139,820
5	Clear Water Transmission (WK-Plein-Blauwgrond)	1,455,975	185,888	485,263	2,127,125
6	Distribution System	8,627,965	1,068,024	2,772,878	12,468,867
7	Miscellaneous Pumping Systems	4,642,258	1,253,080	499,756	6,395,094
8	Rehabilitation Works Republiek	1,489,686	630,808	625,645	2,746,139
9	Rehabilitation Works WK-Plein	1,208,986	410,617	266,942	1,886,545
10	Rehabilitation Works Livorno	531,598	345,509	315,084	1,192,191
	Sub-totals 1 New Works	34,120,418	8,646,150	9,843,031	52,609,599
	sub-totals 1 Rehabilitation Works	3,230,269	1,386,933	1,207,671	5,824,874
	Sub-totals 1 All Works	37,350,687	10,033,083	11,050,702	58,434,473
	Engineering Design	1,850,000	150,000	0	2,000,000
	Construction Supervision 5%	2,702,594	219,129	0	2,921,724
	Sub-totals 2 All Works	41,903,282	10,402,212	11,050,702	63,356,196
	Physical Contingencies 10%	4,109,022	1,041,230	1,185,367	6,335,620
	Totals All Works	46,012,304	11,443,443	12,236,070	69,691,816

17.4 HISTORICAL FINANCIAL ANALYSIS

The financial statements show a reasonable performance of SWM in the past five years. Solvability and liquidity indicators are satisfactory, but profitability remains frail. Yet this performance cannot mask some serious underlying problems.

The assets of the SWM are valued against historical costs. In the particular environment in which the company has to operate, this policy results in overstated profits and de-capitalization. Revaluation of assets will result in a vast increase of the asset value, even if the starting point of such an operation is the official exchange rate.

17.5 FINANCIAL FEASIBILITY

The financial viability of the project has been assessed using two models: the SPMOD-model of the Inter-American Development Bank and the Financial Planning Model of IWACO.

The SPMOD-model is stated in constant US$ prices. The results of this model are straightforward. No matter what scenario will be applied, substantial real rate increases will be necessary to guarantee financial viability. If a profitability break even point should be established, real rate increases should vary from 80% in scenario A to almost 100% in scenario B. If however, a liquidity break even point has to be realized, incidental real rate increases have to be 135 and 139%.

In the Financial Planning Model the required rate increases are even higher due to the application of a price elasticity mechanism in this model. Yet, due to the very high price elasticity, big rate increases result in extremely low demand.
The Financial Planning Model uses a more relaxed criterion than the SPMOD-model. According to this criterion NO annual accumulated liquidity shortages may occur. However, implementing this criterion will require very big rate increases, especially when scenario B will occur.

17.6 TARIFF ANALYSIS AND NEW TARIFF PROPOSAL

The new tariff structure was prepared trying to meet several of the following criteria. First, the tariff has to be fair. Customers have to pay according to the cost they impose on the system. Secondly, the SWM wants to provide water to customers who cannot afford the full cost of the service ('basic need approach'). Thirdly, the schedule has to cover the financial needs of the SWM. An important economic consideration is to use resources efficiently. Finally, the administrative capacity of the organization should be reflected in the new structure.

The new structure is less complicated for SWM's administration. The number of tariff categories is reduced to six. Furthermore, the determination of the annual rental value of the customer's dwelling which is considered as an arbitrary process, is no longer required.
Moreover, the tariff structure does not give any contradictory incentives. In the new system consumption above the minimum consumption level will give all customers an incentive to use water more efficiently.

In the new system regular inflation adjustments will still be necessary. Regular adjustments (annually or bi-annually) are essential to let tariffs keep in line with inflation rates. If inflation compensation does not occur, the principle of cost recovery can be jeopardized.

For the SWM equity considerations are equally important in the design of a tariff structure. In the new tariff structure, considerable attention is given to equity. The system guarantees the lower income groups the use of a minimum consumption level against subsidized tariffs.

The required rate increases for 1992 and 1995 are rather big. It is assumed that the rate increase of 1995 will coincide with the implementation of the new tariff structure.

The existing tariff structure has been evaluated and a new tariff structure has been proposed.
A new tariff structure has to be prepared based on the following criteria. First of all, the tariff has to be fair. Customers have to pay according to the cost they impose on the system. Secondly, the SWM wants to provide water to customers who cannot afford the full cost of the service ('basic need approach'). Thirdly, the schedule has to cover the financial needs of the SWM. An important economic consideration is to use resources efficiently. Finally, the administrative capacity of the organization should be reflected in the new structure.

The new structure is less complicated for SWM's administration. The number of tariff categc reduced to six. Furthermore, the determination of the annual rental value of the customer's dwelling, which is considered to be arbitrary, is no longer necessary.
The tariff structure does not give any contradictory incentives. In the new tariff structure consumption above a certain level will give all customers an incentive to use water more efficiently.

Also in the new system regular inflation adjustments will still be necessary. Regular adjustments (annually or bi-annually) are necessary for the tariff to keep track with inflation rates. If adjustment to inflation levels does not occur, the principle of cost recovery will be jeopardized.

For the SWM equity considerations are equally important in the design of a tariff structure. In the new tariff structure, considerable attention is given to equity. The system guarantees the lower income groups the use of a minimum consumption level against subsidized tariffs.

The necessary rate increases for 1991 and 1995 are rather big. It is assumed that the increase of 1991 will take place according to the old tariff structure. The rate increase of 1995 will coincide with the implementation of the new tariff structure.

17.7 ECONOMIC ANALYSIS

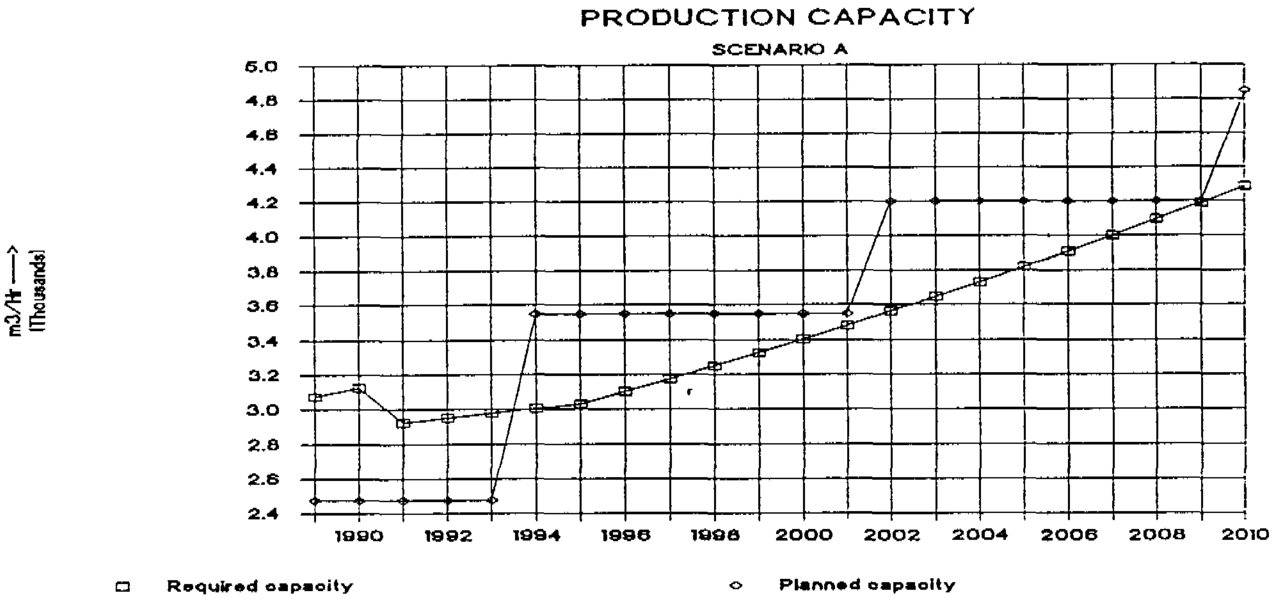
For both scenarios the Consultant has to conclude that the project is not feasible. The net present values and the internal rates of return are not meeting the usual criteria.
This disappointing result is mainly caused by the shadow exchange rate and the price elasticity of non-residential water demand. Because of the enormous gap between the official and the shadow exchange rate and the very high import component of the project, the project costs are extremely high. Yet reduction of the high import component is not possible due to the specific characteristics of the Suriname economy.

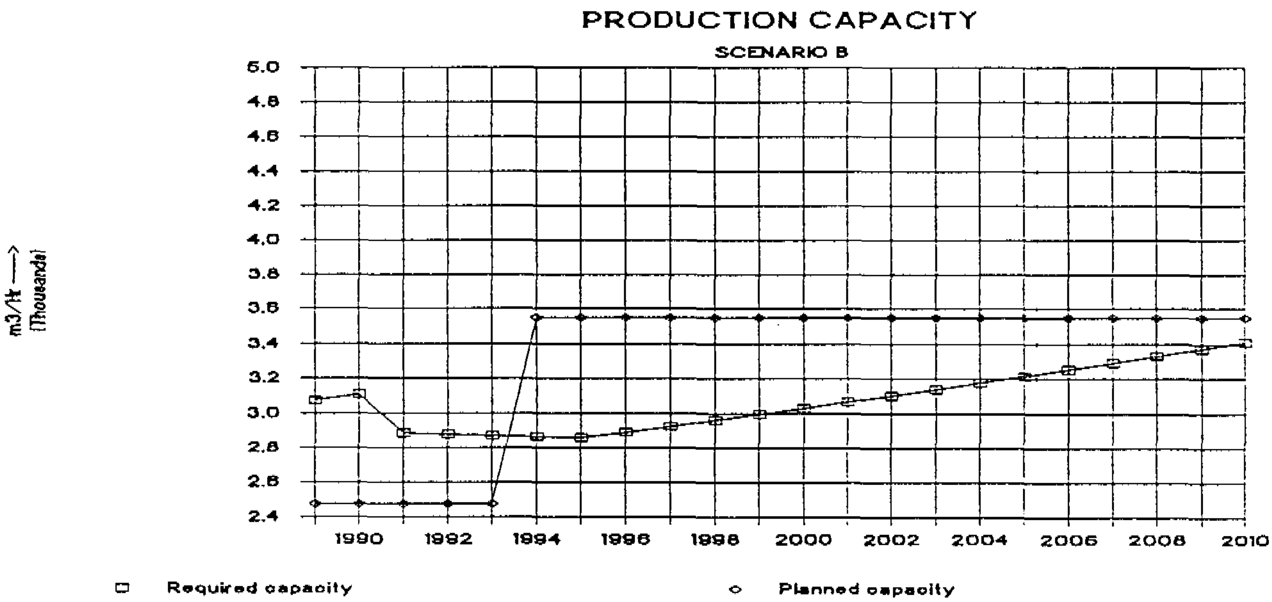
The original assumptions regarding the price inelasticity of non-residential water demand could not be maintained, because the maximum willingness to pay reached very high and unreliable levels when using the SIMOP-model.
However, changes in the price elasticity for non-residential water demand have serious consequences for the feasibility of the Paramaribo Water Supply Project.
A lower price elasticity will reduce the willingness to pay to more realistic proportions, but will also result in negative net present values and very low rates of return.

17.8 SOCIAL FEASIBILITY

The total gross benefits accruing to low-income households are determined by employment benefits and an enlargement of the consumer's surplus. It has been assumed that all employment benefits will accrue to low-income households as many of the unskilled workers which will be employed in the project are presently unemployed.
The gross benefits deriving from the gain in the consumer's surplus in the 'with project' situation is reduced due to the big difference between actual water rates and the willingness to pay in the 'without project' situation.
Depending on the applied scenario, about 40% of the net benefits of the project will go to low-income groups.

FIGURES





NV Surinaamsche Waterleiding Maatschappij

HARRIS
Fredric R. Harris, Inc.,
New York, The Hague

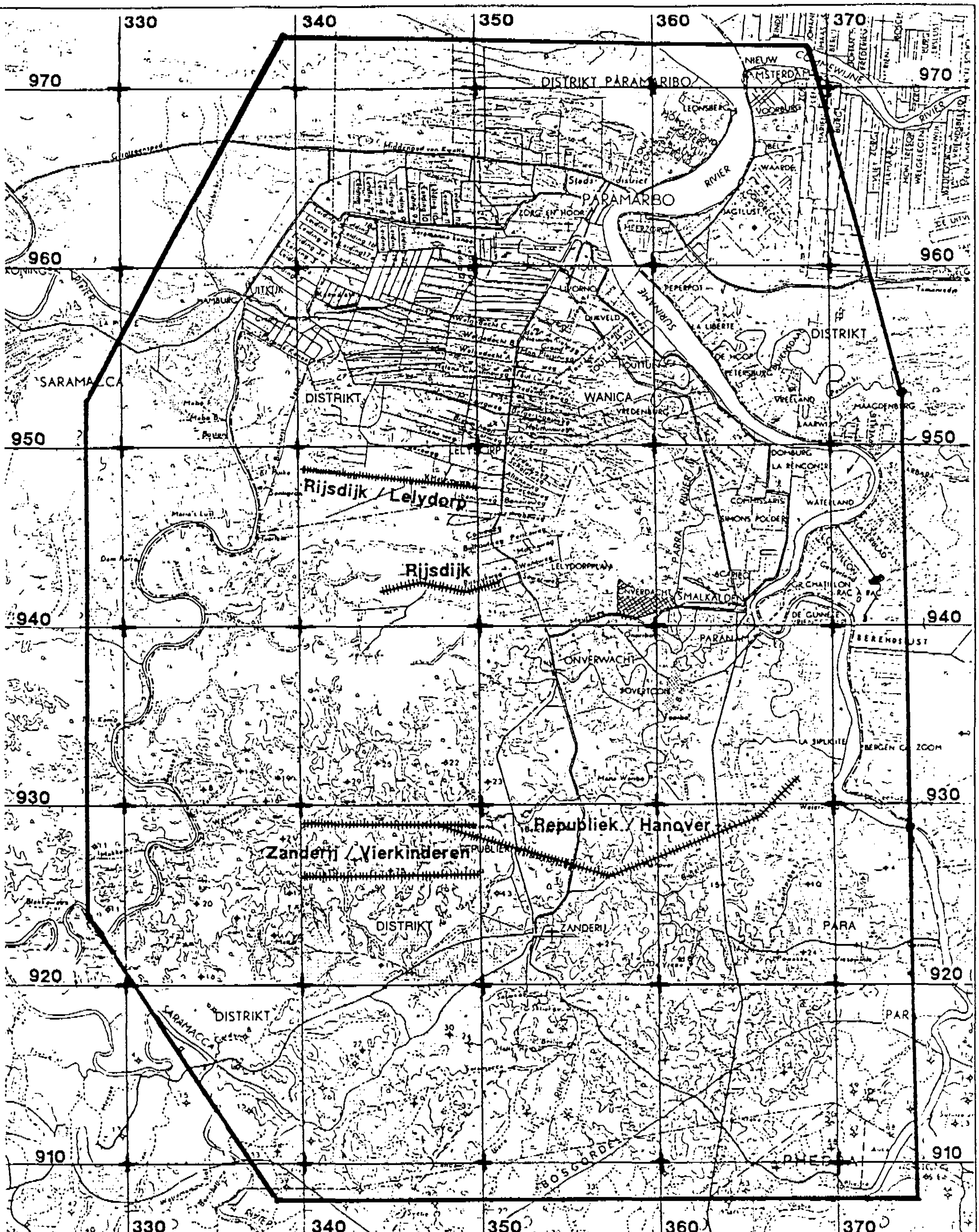
In association with SUNECON–PARAMARIBO

IWACO
Consultants for Water & Environment
Rotterdam, The Netherlands

Water Supply System for Paramaribo & its Metropolitan Area	Date	Drawn	Checked	Approved

Description: Required Overall Production Capacity

Drawing number: 2.1



≋SWM≋ NV Surinaamsche Waterleiding Maatschappij

HARRIS
Frederic R. Harris, Inc.,
New York, The Hague

In association with SUNECON-PARAMARIBO

IWACO
Consultants for Water & Environment
Rotterdam, The Netherlands

Water Supply System for Paramaribo & its Metropolitan Area	Date	Drawn	Checked	Approved
	12 - 1990	NK		

Description: Model area

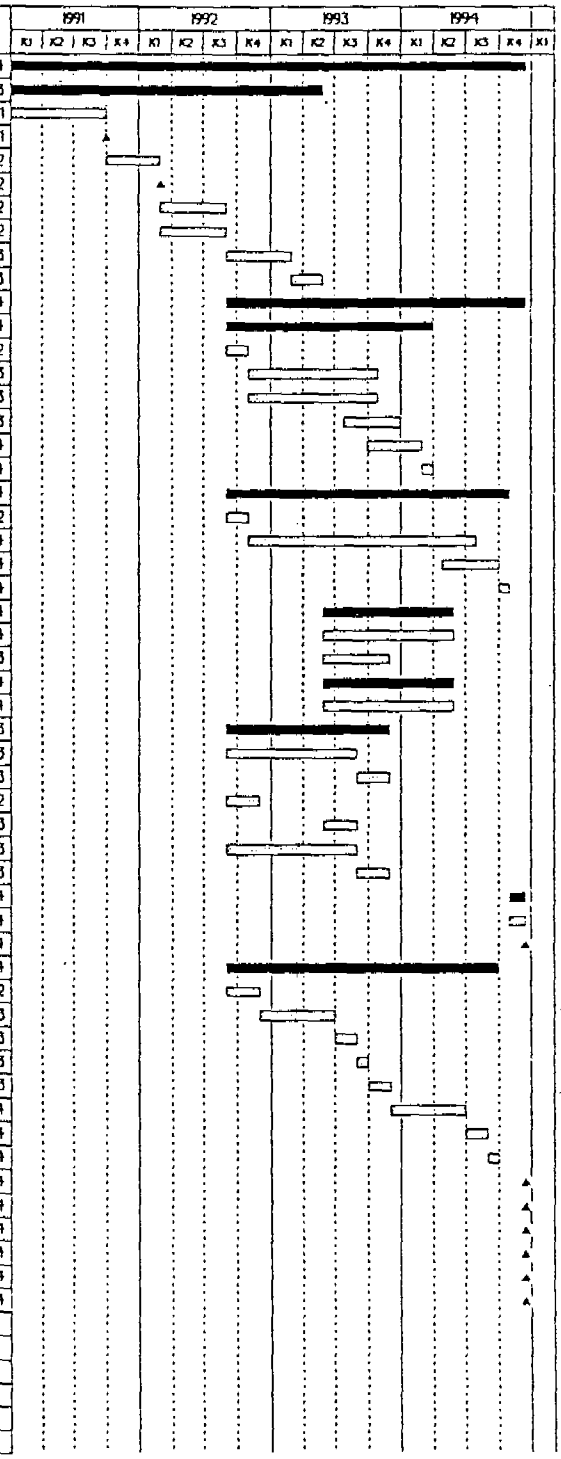
Drawing number: 5.1

Legend :

potential wellfield

PARAMARIBO WATER SUPPLY PROJECT
IMPLEMENTATION SCHEDULE

Taaknaam	Start Datum	Eind Datum
TOTAL PROJECT	01-08-90	15-12-94
GENERAL - PREPARATION	01-08-90	28-05-93
Detailed Design Phase	01-08-90	30-09-91
Funding Requirements	01-10-91	01-10-91
Negotiations Project Financing	01-10-91	28-02-92
Financing Agreement	02-03-92	02-03-92
Tendering Civil Works	02-03-92	31-08-92
Tendering E & M Works	02-03-92	31-08-92
Production E & M Equipment	01-09-92	26-02-93
Shipment E & M Equipment	01-03-93	28-05-93
CONSTRUCTION	01-09-92	15-12-94
WELL-FIELD	01-09-92	31-03-94
Mobilization	01-09-92	30-10-92
Construction of Roads	02-11-92	28-10-93
Drilling Activities	02-11-92	28-10-93
Laying Well-field Header	29-07-93	30-12-93
Installation E & M Equipment	29-09-93	28-02-94
Testing Well-field	01-03-94	31-03-94
NEW WATER TREATMENT PLANT	01-09-92	31-10-94
Mobilization	01-09-92	30-10-92
Implementation Civil Works	02-11-92	29-07-94
Installation E & M Equipment	27-04-94	30-09-94
Testing WTP	03-10-94	31-10-94
CLEAR WATER TRANSMISSION MAINS	01-06-93	31-05-94
Laying WTP - WK Plein	01-06-93	31-05-94
Laying WK Plein - Blauwgrond	01-06-93	29-11-93
DISTRIBUTION MAINS	01-06-93	31-05-94
Laying New Mains	01-06-93	31-05-94
MISCELLANEOUS NEW WORKS	01-09-92	29-11-93
WK Plein Distr. Centre Civil W	01-09-92	31-08-93
WK Plein Distr. Centre E & M W	01-09-93	29-11-93
Blauwgrond Civil Works	01-09-92	27-11-92
Blauwgrond E & M Works	01-06-93	31-08-93
Livorno Civil Works	01-09-92	31-08-93
Livorno E & M Works	01-09-93	29-11-93
COMMISSIONING NEW SYSTEM	01-11-94	15-12-94
Testing	01-11-94	15-12-94
Commissioning	16-12-94	16-12-94
REHABILITATION WORKS	01-09-92	30-09-94
Install Temporary Prod. Unit	01-09-92	30-11-92
WK Plein WTP Civil Works	01-12-92	30-06-93
WK Plein WTP E & M Works	01-07-93	31-08-93
WK Plein WTP Test/Commiss.	01-09-93	30-09-93
Install Temporary Prod. Unit	01-10-93	30-11-93
Republiek WTP Civil Works	01-12-93	30-06-94
Republiek WTP E & M Works	01-07-94	31-08-94
Republiek WTP Test/Commiss.	01-09-94	30-09-94
TAKING OUT OF USE FACILITIES	16-12-94	16-12-94
Bernie's Park (50 m3/hr)	16-12-94	16-12-94
Flora (100 m3/hr)	16-12-94	16-12-94
Leysweg Phase 1 (375 m3/hr)	16-12-94	16-12-94
Leysweg Phase 2 (150 m3/hr)	16-12-94	16-12-94
Lelydorp (250 m3/hr)	16-12-94	16-12-94



NV Surinaamsche Waterleiding Maatschappij

HARRIS
Fredric R. Harris, Inc.,
New York, The Hague

In association with SUNECON–PARAMARIBO

IWACO
Consultants for Water & Environment
Rotterdam, The Netherlands

Water Supply System for Paramaribo & its Metropolitan Area	Date	Drawn	Checked	Approved

Description
IMPLEMENTATION SCHEDULE

Drawing number
Figure 16.1