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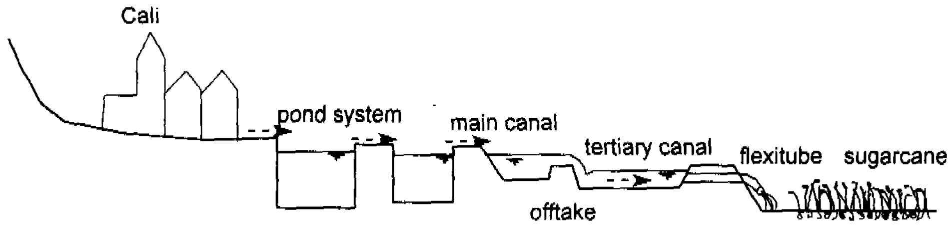

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Wastewater Reuse for the Irrigation of Sugarcane in Cali Region, Colombia

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Preface

This report is a revision of my MSc thesis for the faculty of Civil Engineering, department of Water Management, Environmental and Sanitary Engineering of Delft University of Technology. My thesis project took place from November 1995 to August 1996, including a five months field period in Cali, Colombia. The field period was carried out with assistance of CINARA, the Universidad del Valle and the Convenio IHE-DUT-Univalle.

I like to thank the following people in Cali for their kind cooperation: Miguel Peña, Hernán Materon, Ines Restrepo, Henri Jimenez, Huub Gijzen, and Rob Lloyd. In Delft I am grateful to Prof.ir. R. Brouwer, prof.dr. M.Donze, prof.ir. J.H.J.M. van der Graaf and ir. P. Ankum for their help and guidance.

Meike van Ginneken.

February 1997.

Summary

Wastewater reuse is the process of treating wastewater for beneficial uses, its transportation to the place of use and its actual use. Reuse of wastewater is a way to minimize depletion and contamination of natural water sources, by reintroducing (partly treated) wastewater as an alternative water resource. This project focusses on direct reuse for irrigation of sugarcane.

Antecedents and Objective

The Valle de Cauca is the basin of the river Cauca, and is located in the South-Western part of Colombia, Latin America. The river Cauca is a highly contaminated river, which is anaerobic for a long stretch. The contamination is for a large part caused by the disposal of untreated municipal wastewater from the city of Cali. Parts of the Valle de Cauca suffer from depletion of water sources.

The general objective of this study is to evaluate possibilities for treatment by natural aquatic treatment, transportation, storage and distribution of wastewater effluent from the EMCALI wastewater treatment plant Cañaveralejo in Cali, Colombia to suit irrigation needs of sugarcane cultivation in the Valle de Cauca. A specific objective is to assess whether treatment by a natural aquatic treatment system for large amounts of domestic wastewater is feasible with regard to use of land area, and socio-economically attractive through the yield of profitable products. The second specific objective is to investigate how the use of wastewater influences sugarcane irrigation, and to integrate wastewater considerations into a distribution system.

The Cañaveralejo wastewater treatment plant is the largest to be built in Cali, It will treat 7.6 m^3/s of domestic and industrial wastewater. The primary effluent of the plant is taken as the influent of the reuse scheme. The quality of the primary effluent of the Cañaveralejo plant is estimated based on the design, data on other plants, measurements, and literature data. The required irrigation water quality is based on literature and local circumstances. The irrigation area of the reuse scheme is located on the right bank of the river Cauca.

Design

A design for a reuse scheme of the effluent of the Cañaveralejo plant is made. This design includes a duckweed pond system for a flow of 475 l/s, which is 4% of the wastewater of Cali. An alternative irrigation scheme, which uses the duckweed pond effluent is made with a size of 936 ha, which is 0.5% of the agricultural area of the Valle de Cauca.

A pond system is selected as the most appropriate treatment system, as it guarantees a high performance and is technically feasible. The pond systems consist of four series of ponds, each of which consist of a 1450 m^2 anaerobic pond, a 12,800 m^2 facultative pond, and two 11,200 m^2 maturation ponds, on which duckweed is grown. Total surface area of the pond system is 20 ha.

An alternative subsurface flow wetland design is made. This system turns out to be technically unreliable and more expensive to construct than the duckweed pond system.

The effluent of the pond system is more suitable for irrigation than primary effluent. The pond system removes the following harmful substances from the wastewater: BOD, suspended solids, excess nitrogen, helminth eggs, and trace elements (notably cadmium).

The designed irrigation system is a furrow irrigation system, flexitubes as quaternary canals, and lined canals. The irrigated area is divided into 10 tertiary units, which get water proportionally. Irrigation and drainage systems are separated. Irrigation is intermittent. Overall efficiency is 49%. The scheme is dual-managed. The canal system has a capacity of 978 l/s. The canal system is an upstream control system with fixed structures (weirs).

Water Balance
A surface water balance and a groundwater balance of the complete Valle de Cauca are made. These balances show that there is no permanent overexploitation of natural water resources. Groundwater resources are temporarily overexploited in dry periods, especially in areas where farmers use groundwater for irrigation.

Economic evaluation
The evaluation includes a financial cost-benefit analysis and a qualitative checklist. This checklist judges impacts which are hard to express in financial terms: effectivity, technical feasibility, institutional and regulatory aspects, social impacts, and environmental impacts. An evaluation is made of three alternatives:

1. Reuse with pond system effluent;
2. Reuse with primary effluent;
3. No reuse; activated sludge treatment, and present irrigation scheme;

The evaluation shows that alternative 1 and 3 are feasible alternatives. Alternative 1 has a more favorable score than alternative 3. Alternative 2 is inexpensive, but ineffective.

Conclusions
Reuse of part of the Cañaveralejo effluent for the irrigation of sugarcane of the area between the Rio Cauca and Rio Fraile is an economically feasible and sustainable alternative to decrease effluent disposal problems of Cali. The Cañaveralejo reuse scheme is cheaper to construct, operate and maintain than the combination of an activated sludge treatment and the old irrigation scheme. The quality of the primary effluent of Cañaveralejo plant can serve as a reliable source for irrigation after treatment with a duckweed pond system, without putting at risk public health, crops or soils.

Duckweed pond systems are technical feasible for large quantities of domestic wastewater. The use of subsurface flow wetlands for the treatment of large urban wastewater flows is not yet a feasible option, as knowledge about and experience with functioning, design, operation and maintenance under tropical circumstances are limited. The wetland concept is promising for Colombia, where temperature is constant and high yearround.

The effluent of a duckweed pond system influences sugarcane irrigation by providing nutrients. The wastewater provides part of the nutrient demand of the sugarcane, so less fertilizer has to be applied. Maintenance of canals and structures increases due to weeds.

An improved centralized irrigation scheme is more economical than unimproved individual farm irrigation schemes, with or without wastewater reuse. An improved furrow irrigation system with flexitubes as quaternary canals, larger furrow spacing, and lined canals can halve water demand.

The need for reuse of wastewater in the Valle de Cauca will increase in the coming years. Contamination will be increased by population growth and the rise of the standard of living. Depletion of groundwater and surface water is a growing problem, as water demand for agriculture, human use and industry increases. Reuse of municipal wastewater is, however, only a very small factor in solving depletion problems: only 6% of the water use in the Valle de Cauca is used for domestic uses. Reuse projects are more attractive in areas where presently groundwater is used for irrigation. Pumping costs are higher and depletion a serious environmental problem.

Resumen

Reuso de aguas residuales municipales de cali para el riego de caña de azúcar

Antecedentes y objetivo
Este proyecto se enfoca en el reuso agrícola directo, considerando el tratamiento, transporte y distribución de aguas residuales para riego. El Rió Cauca es una fuente altamente contaminada, especialmente por las aguas residuales de la ciudad de Cali. El Valle del Cauca sufre de un agotamiento paulatino de sus recursos naturales acuáticos y por lo tanto el reuso agrícola de aguas residuales municipales podría ser una solución atractiva para ayudar a solucionar los problemas de contaminación y agotamiento del recurso. Para determinar la factibilidad económica del reuso se necesitan investigaciones más detalladas.

El objetivo general del estudio es evaluar las posibilidades de tratamiento, transporte, almacenamiento y distribución de efluentes de aguas residuales de la futura planta de tratamiento de Cañaveralejo de Emcali. Las tecnologías consideradas en este estudio son de tipo natural y se considerará su potencial para servir las necesidades del riego de cultivos de la caña de azúcar en el Valle del Cauca.

Sistema de tratamiento
Las lagunas de macrofitas flotantes (lemnia) son seleccionadas como sistemas apropiados para el tratamiento, dado que garantizan una alta eficiencia y generan el beneficio adicional de un subproducto rico en proteína con valor económico. El caudal del sistema de tratamiento es el necesario para suplir la demanda de riego del área de estudio y es 475 l/s. El sistema de lagunas se compone de 4 series con un laguna anaerobica, un laguna facultativa, y dos lagunas de materación con con lenteja de agua. El área total de este sistema es 20 ha de las cuales 10 ha se utilizarán para el crecimiento de lemnia.

Esquema de riego
El agua recuperada se utilizará para riego del cultivo de caña de azúcar. El esquema de riego comprende 936 ha sobre la margen oriental del Rió Cauca y se diseña con surcos alternos de riego. El esquema incluye tuberías flexibles para canales terciarios, canales revestidos y reservorios para almacenamiento nocturno. La eficiencia total del sistema de riego es del 49%. El sistema de suministro es "intermitente" y "proportional". Algunas ventajas y desventajas del uso de aguas residuales para suelos y plantas así como aspectos de salud pública son discutidos en este proyecto.

Evaluación económica
Las siguientes posibilidades serán evaluadas:
1. Reuso, sistema de lagunas con lemnia, sistema mejorado de riego;
2. Reuso, tratamiento primario, sistema mejorado de riego;
3. No reuso, tratamiento primario y secundario convencional, sistema mejorado de riego;

Conclusiones
El reuso de las aguas residuales de la planta Cañaveralejo es una solución económica y sostenible para reducir agotamiento y contaminación.
El sistema de lagunas con lemnia no presentan problemas de factibilidad technica cuando se necesita tratar grandes flujos de aguas residuales.
El efluente del sistema de lagunas influye el esquema de riego por la provisión de nutrientos. Menos uso de fertilizantes es necessario.
Un esquema de riego nuevo es mas económica que el esquema decentralizado de hoy. La necessidad de agua disminuye 50%.
El problema de agotamiento y contaminación del agua esta creciendo en el Valle de Cauca y por lo tanto los esquemas de riego con aguas residuales serán cada vez mas factibles.

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1 Introduction

This chapter introduces the subject and antecedents of the study, and defines the objectives and scope of the study. An introduction of the terminology of reuse is given in §1.1. The antecedents of the project (§1.2) lead to the problem definition (§1.3), and the objectives of the study (§1.4). §1.5 describes the scope of the study and lists the limitations and delimitations. The research methods which are used in the study are introduced in §1.6. The final paragraph of this chapter (§1.7) explains the structure of this report.

1.1 Reuse of Wastewater

Human activity upsets the natural water cycle by the use of water sources and disposal of wastewater. The use of water from natural resources can lead to the depletion of both groundwater and surface water sources. Disposal of waste and used water into the natural water cycle causes contamination of natural water sources. The awareness that by depleting and contaminating natural resources, man destroys his environment and ultimately himself, has grown over the years. The World Commission on Environment and Development put it into words in another way (cited in Umali, 1993, p.58): "Development which destroys the natural resources on which it is based is not development". In order to minimize depletion of natural sources one has to increase the efficiency of water use and to change the source to the natural resource that is least sensible for depletion. In order to reduce contamination, wastewater treatment must be introduced or improved.

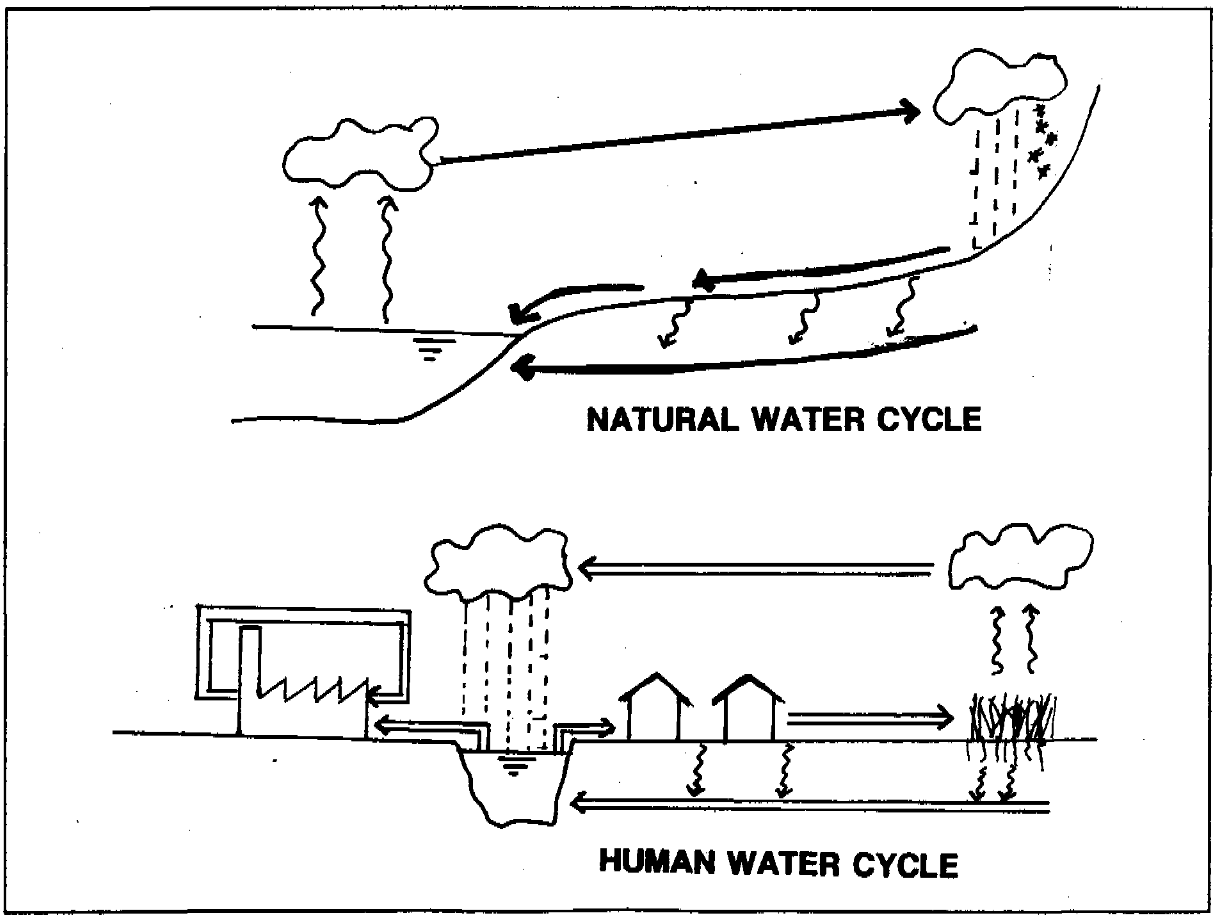


Figure 1: Natural versus human water cycle

Reuse of wastewater is a sustainable way of minimizing both depletion and contamination of natural water sources, by reintroducing (partly treated) wastewater as an alternative water resource. Wastewater reuse systems in effect, mimic the natural water cycle through engineered processes. As the link between wastewater, reclaimed water, and water reuse has become better defined, increasingly smaller recycle loops are possible (see figure 1).

Wastewater reclamation or wastewater reuse is the process of treating wastewater for new uses, its transportation to the place of use and its actual use (Pettygrove and Asano, 1985). A distinction can be made between direct reuse (when there is a direct piped or canal connection of a wastewater effluent to the intake of the new use), and indirect reuse (when water for productive use is abstracted from a natural surface or underground source that is fed in part by the discharge of wastewater effluent).

Reuse can be categorized according to the productive use the water will have after treatment and transportation. The four main categories are:

- agricultural reuse: reuse for irrigation;
- aquacultural reuse: reuse for fish-farming;
- industrial reuse: reuse for industrial use;
- potable reuse: reuse for human consumption;
- household reuse: reuse for household uses.

This project focusses on direct agricultural reuse, the treatment, transportation and distribution of wastewater effluent for irrigation. Agriculture uses 85% of available water in developing countries. Irrigated agriculture should therefore play a major role in solving depletion problems and effluent disposal problems.

Agricultural reuse is the easiest reuse to realize as agriculture can accept lower quality water than domestic and industrial users. In quantitative terms, the volume of domestic wastewater available for reuse by irrigated agriculture is negligible when compared with the overall volume of water used for irrigation. However, the potential environmental and social impacts of agricultural reuse of wastewater are so important, that the need for sound planning exceeds the relatively small quantities and areas involved (Ayers and Westcot, 1985).

Agricultural wastewater reuse is a multi-disciplinary process, involving sanitary, irrigation and agronomic engineering as well as hydrology and economy. Sanitary engineers evaluate and minimize public health risks, and design wastewater treatment. Irrigation engineers evaluate operational risks and design distribution systems. Agronomists evaluate risks to crops and calculate water needs, given a certain water quality. Hydrologist evaluate the availability of water resources, the consequent need of reuse, and the influence it will have on the natural water cycle. Economists evaluate costs and economic feasibility of reuse projects. Because wastewater contains impurities careful consideration must be given to protection of public health, prevention of damage to crops and soils, and prevention of nuisance conditions during storage and operation. These effects are normally manageable if associated problems with impurities are understood and accounted for (Ayers and Westcot, 1985).

1.2 Antecedents of the Project

In recent years several Latin American countries, notably Mexico, Colombia, and Chile, have experienced elevated levels of economic growth in the commercial and financial sectors. Approximately US$ 4 * 10^9 will be spent on water supply and wastewater treatment in the Andes region in the next four years (Gijzen, IHE, pers.comm.).The economic growth has led to an increase of government investment in the provision of public services, especially in water supply and sewerage. Reasons for the poor coverage in many regions are an inability to pay for expensive conventional technology, a lack of infrastructural and institutional development, a scarcity of human resources to facilitate operation and maintenance, and a lack of community involvement in development projects.

The situation in Colombia is similar to the general South-American situation. In March 1995, the National (Colombian) Planning Department in coordination with the National Council of Social and Economic Policy launched the "Water Plan" for the period 1995-1998 with the broad aims of improving the coverage, functioning, and institutional management of the sector and the overall goal of achieving 90% coverage of water supply and 77% coverage of sewer systems by 1998. A further objective of the "Water Plan" is to reduce the negative effects of wastewater discharges on all rivers and receiving bodies of water through the effective treatment of wastewater. The plan intends to improve the quality of life for those people living near the receiving water course, and also of reducing the cost of the treatment of potable water, and providing a valuable natural resource for reuse with a minimum risk to public health (Departamento Nacional de Planeacion, 1995).

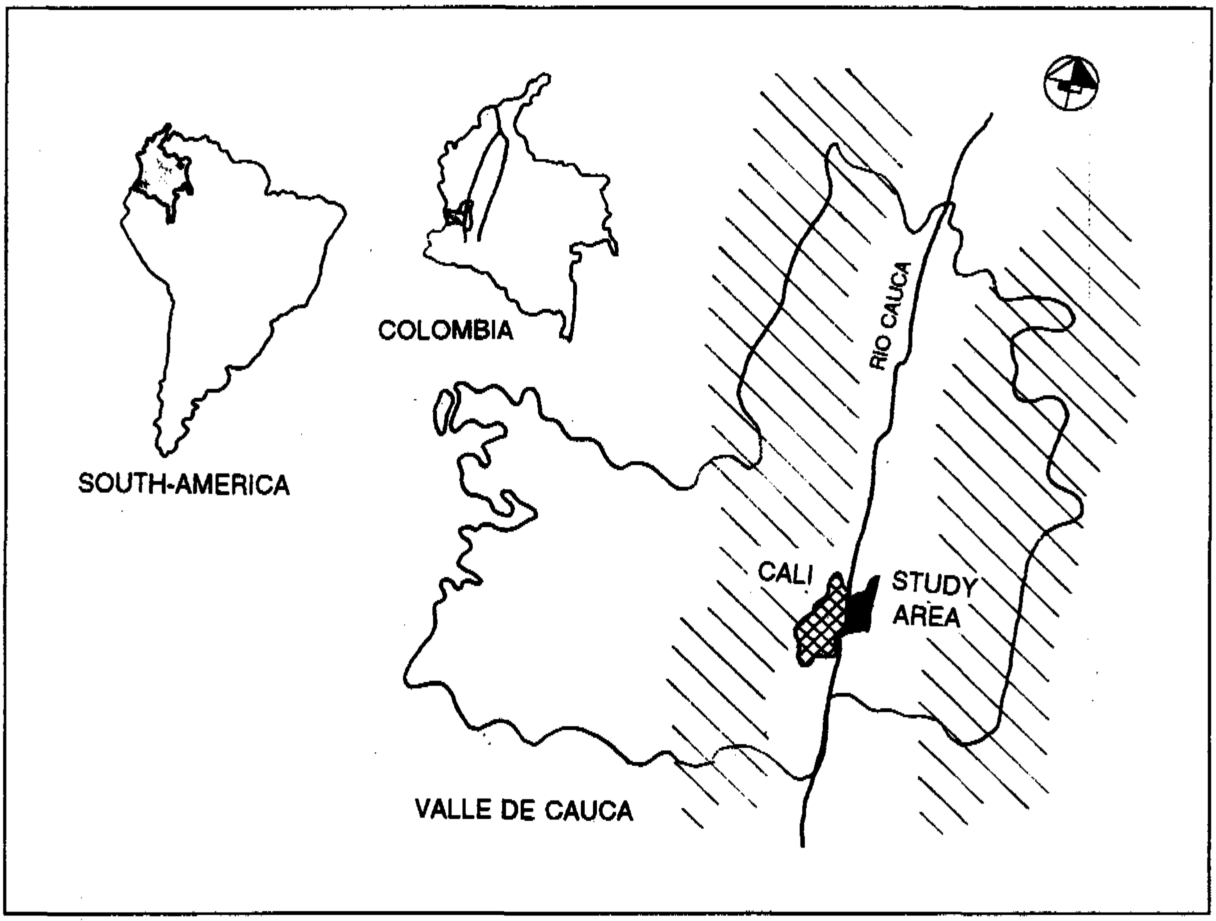


Figure 2: Map of the Valle de Cauca

This study focusses on the Valle de Cauca. Valle de Cauca is a province in the South Western part of Colombia, Latin America, and is located between 3° and 5° North and 75° and 77° West (see figure 2). Valle de Cauca also is the name of the basin of the river Cauca. In this report the name Valle de Cauca refers to this part of the Cauca river basin which is located within the Valle de Cauca province. The borders of this area are the Cordillera Occidental on the Western side, the Cordillera Central on the Eastern side, the cross section at Puenta de Balsa in the South and the cross section at Puenta Anacaro in the North. The surface area of the Valle de Cauca is 14,800 km^2 of which 3,370 km^2 is flat land and 11,430 km^2 is mountainous. Total population is 3,158,000 of which more than half lives in Cali, the province's capital.

The Valle de Cauca is a rich agricultural area: 310,000 ha is used for agriculture, of which 185,000 ha for sugarcane cultivation (Alzate, 1994; CONE, 1995). The water demand of the sugarcane farmers is high and pumping costs are high, much effort is put into decreasing water demands. Water is obtained both from surface water (the River Cauca and its attributive streams) and groundwater. The sugarcane farmers are united in ASOCAÑA, research is carried out by CENICAÑA, the national sugarcane research institute.

Cali is the capital of the Valle de Cauca province the second city of Colombia. Its population (1994) is 1,844,000 (Guzman, 1995). The population is estimated to grow to more than 3,000,000 by 2020 (Banguero and Castellar, 1993). Of Cali's population 94% is connected to the water supply system and 80% to sewerage (Alzate, 1994).

The river Cauca the main stream in the Valle de Cauca is a highly contaminated river. At present practically all domestic wastewater of Cali city is discharged untreated on the River Cauca, this discharge accounts for 72% of the BOD (biochemical oxygen demand) load of the river. Other polluters are industries, the municipality of Yumbo, and several paper mills. Downstream of Cali a 80 km long stretch of the river is anaerobic and therefore creates a habitat for insects, mollusca and bacteria. Due to the construction of a hydroelectrical dam upstream at Salvanija, discharges have dropped and water scarcity has grown in the Valle de Cauca.

EMCALI is the municipal works department responsible for wastewater collection and treatment. EMCALI planned the construction of 4 wastewater plants in the future, the largest will be the Cañaveralejo plant treating a discharge of 7.6 m^3/s. Some of these plants are not to be finished in the next 25 years and even after construction of the plants the wastewater disposal problem of Cali will not yet be solved: the plants will only partly treat the wastewater and by that time the city's population will have grown.

The CVC is the provincial water resources body and hence is responsible for control of water contamination and use of natural water sources. The contamination of natural water bodies is regulated in law 1594 (Republica de Colombia Ministerio de Salud, 1984) and Acuerdo No.14 (CVC, 1976), which up till now has not been enforced. Last year the CVC introduced a tax on water use (CVC, 1995), which is very low at the moment but will be increased in the future. There are no special laws for wastewater reuse, and it is not clear if and when effluents for reuse will have to comply to the laws mentioned above.

1.3 Problem Definition

The river Cauca is a highly contaminated river, which is anaerobic for a long stretch. The contamination is for a large part caused by the disposal of untreated municipal wastewater from the city of Cali into the river.

The Valle de Cauca suffers from depletion of water sources. The use of surface water is regulated by law, but disputes between municipal and agricultural users occur often. There is a discussion whether the groundwater in the region is overexploited. New water taxes are imposed on the use of both surface and groundwater.

The Valle de Cauca is one of the richest region of Latin America, but even here economically feasible solutions for the problems stated above should be found.

1.4 Objectives

Agricultural reuse of municipal wastewater could be an attractive solution to help to solve the contamination and the depletion problem in an economically feasible way. In order to determine the effects of reuse as well as the economical feasibility, further investigation is required.

The general objective of this study is to evaluate possibilities for treatment by natural aquatic treatment, transportation, storage and distribution of wastewater effluent from the EMCALI wastewater treatment plant Cañaveralejo in Cali, Colombia to suit irrigation needs of sugarcane cultivation in the Valle de Cauca.

A specific objective will be to assess whether treatment by a natural aquatic treatment system for large amounts of domestic wastewater is feasible with regard to use of land area and socio-economically attractive through the yield of profitable products.

The second specific objective will be to investigate how the use of wastewater influences sugarcane irrigation and to integrate wastewater considerations into a distribution system.

1.5 Scope of Study: Treatment - Transportation - Storage - Distribution

Delimitations are limitations on the research design that the researcher imposes deliberately. Limitations are restrictions over which the researcher has no control (Huckin, 1991).

1.5.1 Limitations

The following limitations are imposed by the situation in Cali:

- The state of EMCALI wastewater treatment plants in Cali: The "Planta UASB Vivero Municipal" is in operation but not functioning properly, the "Planta Cañaveralejo" and the "Planta UASB Rio Cali" will be commissioned before 2001 and "the "Planta UASB Sur" is foreseen around 2011.
- National and regional legalization.
- The topography of the Valle de Cauca, including areas used for domestic and industrial purposes, the location of rivers and streams and elevation levels.
- The social circumstances of the farmers, their knowledge and skills.

1.5.2 Delimitations

Delimitations are limitations on the research design that the researcher imposes deliberately. These restrictions include the treatment plant for effluent, the study area for irrigation, and other delimitations for the treatment system and the irrigation scheme.

Treatment Plant for Effluent

The Cañaveralejo wastewater treatment plant effluent is selected to serve as the influent for the reuse scheme. Construction of the Cañaveralejo plant will most probably start in January 1997 and involves large expenses. EMCALI has not yet thought about post-treatment options, but is aware it has to consider these in the coming years. As a result EMCALI is very interested in topics relating to the Cañaveralejo plant. Effluent data of the future plant are available.

Other treatment plants are less suitable for a reuse study. The Vivero plant is the only plant in operation at the moment, but it is a pilot scale plant and a study into reuse possibilities would not represent full-scale reuse of wastewater. The Rio Cali plant and the Planta Sur design are not yet ready, and no data exist on the effluent quality.

Study Area for Irrigation

The reclaimed wastewater is used to irrigate sugarcane on the right bank of the river Cauca in the Valle de Cauca. The prevailing slope in the Valle de Cauca is from the South (high) to the North (low), so from the Cañaveralejo plant only an area to the North can be irrigated by gravity; the railway line can serve as a border. The choice of the extent of the study area depends on the availability of data. The three main categories of data necessary to plan an irrigation scheme are climatological data, soil data and topographical data.

Based on the availability of climatological and agronomic data the Juanchito-Guanabanal-Palmaseca is chosen (see figure 3). This area is well documented in a thesis about climate and soils (Aristazabal and Zambrano, 1993). The Juanchito-Guanabanal-Palmaseca region is situated on the East bank of the Cauca river, at an altitude of 950 m. Juanchito, Guanabanal and Palmaseca are corregimientos, jurisdictional zones, of the municipality of Palmira. The area is bordered in the East by Palmira, in the West by the Rio Cauca and Cali city, In the North by the corregimiento of Vereda Matapalos and in the South by the municipality of Candelaria.

There are no elevation data on the area South of the Cali- Palmira road and East of the Rio Fraile. The area South of the Cali-Palmira road, East of the Rio Fraile will not be included in further investigation.

Data on the area East of the Rio Bolo/Rio Gauchal and North of the Cali-Palmira road are incomplete. The area is very distinct from the rest of the Juanchito-Guanabanal-Palmaseca area, as there are serious salinity problems. The objective of this study is not to look into various irrigation problems; in order to be able to focus on reuse issues, this area will not be taken into account.

The study area will be the area between the Rio Cauca and the Rio Fraile/Rio Gauchal on the East. The South border will be the railway Cali-Palmira, and in the North the area will be bordered by the corregimiento of Vereda Matapalos. The surface area of the study area is 1237 ha.

Other Delimitations Treatment System

Other delimitations regarding the treatment system are:

- Primary effluent is considered.
- A macrophyte-based wastewater treatment system treats the wastewater prior to irrigation. Macrophyte-based treatment systems are defined as discharging natural wastewater treatment systems in which aquatic macrophytes (wetlands plants, which are adapted to growing conditions in water saturated soils) have a key function in relation to the cleaning of wastewater (Brix, 1993). The treatment system should meet local community interests and yield interesting products (duckweed, reed etc). Macrophyte-based treatment concepts which meet these requirements are: hyacinth and duckweed ponds and constructed wetlands (both free water surface and subsurface flow). Submerged macrophyte-based systems, multistage systems and vertical-flow systems will not be discussed.
- The location of the pre-irrigation treatment can be on the right bank of the river Cauca. Transportation to the pre-irrigation site and storage of water at the EMCALI plants is not designed. Transportation is taken into consideration when analyzing the costs and benefits of the system.

Other Delimitations Irrigation System

Restriction for the irrigation scheme are:

- The design of the irrigation system will be "from scratch", as revising of the old system is not practicable in the scope of this thesis. The design will include transportation from the pre-irrigation site, any storage and distribution. The irrigation scheme is productive, or crop-based irrigation: the objective of the irrigation management is to maximize the return per unit area. The irrigation scheme has to function by gravity.
- Drainage is not designed.

- It is assumed that the complete agricultural area is used for sugarcane. In reality over 90% is used for sugarcane. Other crops include annual crops and pastures.
- The use of wastewater for irrigation may not cause a drop in sugarcane yield.

1.6 Research Methods

The objectives stated above will be fulfilled using mainly existing data. No new data will be produced via practical and experimental work, except a series of electric conductivity measurements in the raw Cañaveralejo wastewater.

Computer programs used include:

- Profile, a computer program to calculate the profile of a trapezoidal canal using Strickler-Manning resistance formula. Version 2.0 of this program made by the Center of Operational Watermanagement of Delft University of Technology was used.
- Cropwat, an irrigation planning and management tool, written by the Land and water Development Division of the Food and Agricultural Organization of the United Nations was used to calculate irrigation needs. Version 5.7, of October 1991 was used.

1.7 Structure of Report

This report focusses on the reuse of municipal wastewater of Cali city, collected by EMCALI, for irrigation of sugarcane cultivation at the East bank of the river Cauca. The study consist of three parts:

1. a theoretical part on wastewater reuse; water criteria for irrigation; natural aquatic treatment systems; generation of criteria set for feasibility;
2. a design for the reuse of the effluent of the Cañaveralejo plant;
3. conclusions and recommendations.

The first theoretical part contains two elements: background from literature in chapter 2 and the generation of a criteria set in chapter 3. Chapter 2 focuses on themes related to the topic of this study and includes a brief history of wastewater reuse (§2.1); an introduction into wastewater characteristics (§2.2); a more elaborate part on irrigation water quality requirements (§2.3); and a discussion about natural aquatic treatment systems (§2.4). Chapter 3 describes the methods and criteria, which are used further on to evaluate the feasibility of the reuse alternative. Methods include a cost-benefit-ratio (§3.1), and a qualitative checklist (§3.2). §3.3 discusses the criteria used for the checklist.

The second part (chapter 4, 5 and 6) of this study is a design of the reuse scheme of wastewater of the Cañaveralejo plant and is the core of the study. Chapter 4 is a selection and a predesign of a treatment system for the effluent before irrigation. The present wastewater is compared to irrigation water criteria (§4.1). A treatment system is selected (§4.2), and designed (§4.3). The pre-irrigation system serves as the water resource for the design of an irrigation system in chapter 5. Present irrigation practices are discussed in §5.1. A 936 ha irrigation scheme is designed, including transportation from the treatment system, storage and distribution (§5.2 - §5.6). Chapter 6 gives an outline of the costs and benefits and other effects of implementation of both the pre-irrigation system and the irrigation system, based on the criteria generated in chapter 3.

The final part of the study (chapter 7) is a chapter of conclusions (§7.1) and recommendations (§7.2). After chapter 7 follows a literature list and appendices.

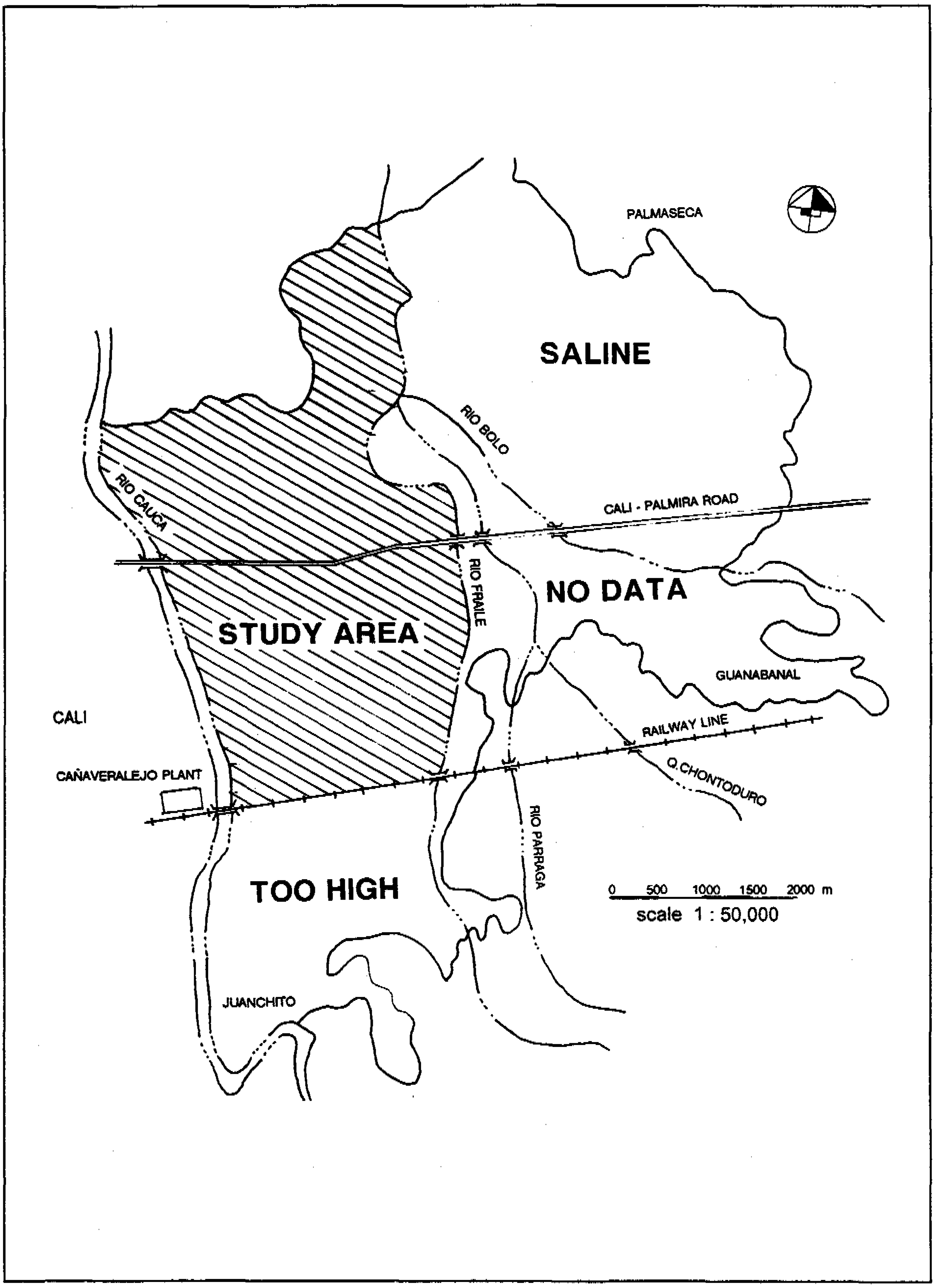


Figure 3: study area

2 Wastewater Reuse for Irrigation using Macrophyte-based Treatment Systems: Theoretical Backgrounds

This chapter gives a short literature review on the history of reuse, municipal wastewater, irrigation water quality guidelines, and macrophyte-based treatment systems. The first paragraphs (§2.1) describes the history of reuse from ancient times to recent years. The characteristics of municipal wastewater and its suitability for irrigation are discussed in §2.2. Irrigation water quality guidelines include three categories: protection of public health risks (2.3.1), prevention of damage to crops and soils (§2.3.2), and prevention of nuisance (§2.3.3). The final paragraph (§2.4) discusses four macrophyte-based wastewater treatment systems: hyacinth ponds, duckweed ponds, free water surface wetlands, and subsurface flow wetlands.

2.1 Brief History of Reuse

The ability of normal, porous aerated soil to purify water as well as the beneficial effects of nutrients in wastewater upon soils have been known for a long time. Use of wastewater for crop irrigation in western civilizations can be traced as far back as ancient Athens. However, the difference between using sewage to irrigate crops and using land to treat sewage has only been recognized in the twentieth century (Dean and Lund, 1981).

During the second half of the nineteenth century there was a considerable increase in the number of reuse schemes in the United States and in Europe, where pollution of many rivers had reached unacceptable levels. Disposal of wastewater on land was the only feasible means of treatment available. Reuse was practiced at so-called sewage farms, with a primary objective of waste disposal, however crop production benefitted from the nutrients present in the wastewater. Some of these nineteenth century sewage farms are still in use today (Hespanhol and Prost, 1994).

A number of reasons led to a significant reduction in the use of wastewater for irrigation. The increase in the volume of wastewater as a consequence of the growth of cities made use for irrigation less feasible, because of the large areas of land needed. The development of wastewater systems at the beginning of the twentieth century created alternative treatment methods. The interest in use for irrigation lowered due to the scientific research provided by Pasteur and Koch revealing that microbes present in excreta cause disease transmission. After Pasteur's discovery hygienists have been promoting the need for a total elimination of pathogens from the human environment. The lack of enforcement on water pollution control caused mismanagement of sewage farms. Furthermore the economic benefits gained from reuse did not balance the increased public perception of the risks involved (Hespanhol and Prost, 1994).

Only in regions were there was a great necessity to meet growing water needs, such as the arid western and southwestern states of the USA, reuse projects were developed in the first half of the twentieth century. In particular in California wastewater (first untreated, then treated in septic tanks) was used from 1912 on for irrigation purposes.

In the past two decades the use of wastewater for crop irrigation has been revived in the arid and semi-arid regions of the world in response to the need for alternative sources of water to increase local food production. The number of wastewater reuse projects is still increasing today. Even in water-plentiful areas reuse of effluent is gaining importance as a beneficial water conservation measure. In many parts of the developing world, wastewater reuse is carried out informally where the obvious benefits to the crops have been recognized. Wastewater reuse is not limited to developing countries, however, as many reuse schemes are found in Australia, Europe, and the USA. Mexico is probably the largest user of wastewater for irrigation in the world: as no conventional wastewater treatment can be economically provided to the flow of about 80 m^3/s of wastewater from Mexico City, it is stored in reservoirs and used for crop irrigation. Mexico has established and is strictly enforcing a policy of crop restriction as a measure for health protection (Hespanhol and Prost, 1994).

The effects of wastewater reuse on public health, crop yield, soil properties and irrigation operation continue to be subject to discussion, although some consistent sets of criteria have been generated (Ayers and Westcot, 1985, Shainberg and Oster, 1978). It had become clear that the "germ free" approach, used by early hygienists is impossible to be achieved for practical reasons. Recent public health regulations have been based on pathogen-host relationships and epidemiological evidence of disease transmission caused by the practice of reuse: helminth eggs are now regarded as the main actual public health risk of wastewater irrigation (Hespanhol and Prost, 1994). Meanwhile scientific literature has reported the results of many studies indicating the beneficial effects of nutrients contained in municipal wastewater, which can be of significant economic benefit in irrigation. Moreover possible harmful influence of irrigation water quality on soil properties and crop growth were discovered, like salinity and sodicity. Recently considerable attention is given to environmental aspects of water quality, including the possible presence of minute amounts of potentially harmful substances, such as trace elements and trace organics (Page and Chang, 1985a, 1985b).

2.2 Municipal Wastewater Characteristics and Suitability for Irrigation

Sewage or municipal wastewater is the spent water of a community. It contains pollutants, consisting mainly of faeces, urine and sullage; and is approximately 99.9% water and 0.1% solids. Municipal wastewater is composed of domestic wastewater, industrial wastewater, infiltration water and, in combined sewer systems, stormwater run-off. The composition of untreated wastewater and the subsequently treated effluent depend upon the composition of the municipal water supply, the number and type of commercial and industrial establishments, and the nature of the residential community.

Due to changing water consumption patterns, increased reliability of water supply and low water prices, wastewater flows have increased in many parts of the world. The volume of domestic wastewater plus infiltration-inflow, excluding industrial wastewater and stormwater runoff, generated in a community on a per capita basis varies from 0.19 to 0.57 m^3/day (Asano et al., 1985). Wastewater flows to a treatment plant can vary widely in quantity. Several cycles can be recognized. The short-term variations in wastewater flows observed at municipal wastewater treatment plants tend to follow a diurnal pattern: flow is low during the early morning hours, when water consumption is lowest, an higher during the afternoon and the evening. In combined sewer systems flow peaks occur due to stormwater runoff. Long-term, seasonal variations in flow may occur in holiday resorts and university campuses. Although mixing and retention time in treatment system depress peaks, variation should be taken into account when using the effluent: storage is required to guarantee a consistent flow (Asano et al., 1985; Van der Graaf, 1995a).

Water quality conceptually refers to the characteristics of a water supply that will influence its suitability for a specific use, and is defined by certain physical, chemical and biological characteristics. Wastewater quality generated by a community varies according to the time of day, the standard of living, the season, and the degree and nature of industrialization in the area in question. Raw wastewater consists the following substances (see table 1):

- Biodegradable organics; caused by domestic, commercial and industrial sources and most commonly expressed in biochemical oxygen demand (BOD) or chemical oxygen demand (COD);
- Solids; ranging from large floatable objects to small invisible particles. A measure for small particles is the concentration of total suspended solids (TSS);
- Nutrients; particularly total nitrogen (TN) and total phosphorus (TP) content of wastewater,

caused by domestic and agricultural wastes as well as natural runoff;

- Pathogens; Domestic wastewater contains a large number of micro-organisms, some of which are pathogenic. The easily identified coliform group of bacteria is used to indicate the presence of other pathogens which are difficult to detect and identify. Helminth eggs are measured separately;
- Micropollutants; These include heavy metals, mainly deriving from industrial wastes and pesticides. Wastewater can include a wide variety of micropollutants in various concentrations;
- Salts; The principal salt present is sodium chloride. Sources include food, water softeners, and salt that is originally present in water supply.

Wastewater quality data measured and reported are mostly in terms of gross pollutant parameters (e.g. BOD, COD, TSS) that are of interest in water pollution control. In contrast, the water characteristics of importance in irrigation are specific chemical elements and compounds that affect plant growth or soil permeability and are not often measured or reported by wastewater treatment agencies as part of their routine monitoring program. As a result it is often necessary to sample and analyze the wastewater for those constituents that define the suitability of the water for irrigation. For purposes of planning, in the absence of actual effluent data, the composition can be estimated from data of the water supply quality and data presented in literature (Asano et al., 1985).

Table 1: Literature values of raw wastewater quality

pollutant	concentration	source
BOD	220 - 290 mg/l	D'Itri,1981; Metcalf & Eddy, 1991; Van der Graaf, 1995a
TSS	220-260 mg/l	D'Itri,1981; Metcalf & Eddy, 1991
TN	30-80 mg/l	D'Itri,1981; Metcalf & Eddy, 1991; Van der Graaf, 1995a
TP	5-15 mg/l	D'Itri,1981; Metcalf & Eddy, 1991; Van der Graaf, 1995a
coliforms	1e7-1e9 /100 ml	Arthur, 1983; D'Itri,1981; Metcalf & Eddy, 1991; Mara et al., 1992
helminth eggs	10-10000 eggs/l	Metcalf & Eddy, 1991; Mara et al., 1992
electric conductivity	0.2-1.3 dS/m	Dean and Lund, 1981; D'Itri, 1981; Metcalf & Eddy, 1995

The general objective of wastewater treatment is to improve water quality to such an extent that it is suitable for its safe disposal into the environment (groundwater or surface water) or its intended reuse. Municipal wastewater treatment consists of a combination of physical, chemical, and biological processes and operations. General terms used to describe different degrees of treatment, in order of increasing treatment level, are preliminary, primary, secondary and advanced or tertiary treatment. Preliminary treatment operations include coarse screening and comminution of large objects and grit removal by sedimentation. Primary treatment removes settleable organic and inorganic solids by sedimentation, and scum by skimming. Secondary treatment involves the removal of biodegradable dissolved and colloidal organic matter using biological treatment processes. Secondary treatment can either be performed by high-rate, mechanized treatment processes or by low-rate, natural processes. Advanced or tertiary treatment is any physical, chemical, or biological treatment process used to accomplish a degree of treatment greater than secondary treatment. Usually it implies removal of nutrients and a high percentage of suspended solids. Salinity, which largely determines the suitability of a wastewater for irrigation, is not reduced substantially in most wastewater treatment systems: in some natural systems salinity may even increase as a result of evaporation (Asano et al., 1985).

The use of municipal wastewater for irrigation should not cause harmful side effects, when treated sufficiently and managed well. Some degree of treatment must be provided to raw municipal wastewater before it is suitable for irrigation. Pre-irrigation treatment of wastewater is practiced for the following reasons: protection of public health; prevention of damage to crops and soils; and prevention of nuisance conditions during storage and operation (Asano et al., 1985). Feachem et al. (1983) stress that waste stabilization ponds, conventional treatment followed by maturation ponds, land application, or sand filtration are the only treatment processes that produce an effluent that can be reused for irrigation without public health problems. The level of pre-irrigation treatment required for irrigation, from an agricultural point of view, depends on soil characteristics, the crop irrigated, and the type of distribution and application system.

2.3 Irrigation Water Quality Requirements

Guidelines are a management tool; they are intended to provide background and guidance for making risk management decisions and are not to be confused with legal standards. Evaluation must be done in terms of specific local conditions of use and farm management ability of the water user (Ayers and Westcot, 1985). Guidelines for evaluation of water quality for irrigation emphasize the reasons for pre-irrigation treatment mentioned above. They can be categorized in three categories:

- Protection of public health.
- Prevention of damage to crops and soils.
- Prevention of nuisance conditions during storage and operation.

This paragraph will discuss all three categories of guidelines and will use a problem-solving approach: guidelines will be followed by suggestions on management alternatives to overcome potential problems (Ayers and Westcot, 1985).

2.3.1 Protection of Public Health

Wastewater contains the disease causing agents of infected humans, such as excreted bacteria, viruses, protozoa, and helminths. Faecal-oral transmitted diseases (including amoebic dysenteries, cholera, giardiasis, and poliomyelitis) and water-based diseases are caused by pathogens transmitted in human excreta. Wastewater use for irrigation can cause two major public health risks: consumers can be exposed to risks by eating (raw) products and workers can be exposed when working with contaminated water (Feachem et al., 1983).

Two indicators are used to assess public health risk in wastewater to be used in irrigation: faecal coliforms and helminth eggs. Because pathogens in water are relatively few in number and difficult to isolate, the nonpathogenic faecal coliform group of bacteria, which is more numerous and easily tested for, is used as an indicator of the presence of viruses, bacteria and protozoans in treated water. Helminth eggs are more persistent than faecal coliforms, and can survive up to two months in an aquatic environment. Accordingly faecal coliforms cannot be used as an indicator for their presence, and the presence of helminth eggs has to be investigated separately.

Guidelines for wastewater reuse have changed several times in the past and even today are still subject to discussion. In 1918 the Californian State Health Department set quality criteria for irrigation. After several revisions this legislation is one of the most complete and restrictive in use today (State of California, 1978). The strictness of these guidelines regarding the amount of coliforms (2.2 per 100ml) has been subject to discussion over the past 20 years, but still many countries have adopted the same criteria with little or no adaptation (Hespanhol and Prost 1994, Mara 1995). In 1971 the WHO Meeting of Experts on the Reuse of Effluents suggested 100 total coliforms per 100 ml for irrigation of vegetables eaten cooked. Epidemiologists and public health experts agreed that the actual risk associated with irrigation with treated wastewater is much lower than previously estimated and that early standards, particularly in respect of bacterial pathogens, were unjustifiably restrictive. Helminth eggs were regarded as the main actual public health risk associated with wastewater irrigation (Hespanhol and Prost, 1994). In 1987 the Scientific Group on Health Guidelines for the Use of Wastewater in Agriculture and Aquaculture once again reviewed the guidelines. In their guidelines the only coliform norm left is for irrigation of crops likely to be eaten uncooked, of sports fields, and of public parks. The 1987 WHO guidelines are included in table 2 appendix VII.

There are four main methods of protecting the public health of a population from the risks associated with wastewater reuse (Lloyd, 1995):

- wastewater treatment: the removal of pathogens from the water before application and exposure is the most reliable way of reducing public health risks. The amount of treatment needed is expressed in the WHO application guidelines (table 2 appendix VII);
- crop restriction: crops eaten raw will cause a higher risk of infection for consumers than cereal crops, industrial crops (such as cotton and sisal), food crops for canning, fodder crops, pasture, and trees;
- control of wastewater application: careful choice of the irrigation system used can help to control the risk of wastewater reuse. The exposure of workers is less when using an automatic system and irrigation with wastewater can be stopped several weeks before harvesting;
- human exposure control: not only for workers but also for their families; the crop handlers who are involved with processing; the consumers; and those people living in the immediate environment of the irrigation. There are a number of ways of exposure control: protective clothing for workers, positioning of sprinklers at least 50-100 m from houses and roads to prevent wetting passers-by, localization of canals within the fields and not at the borders, information about the location of the fields where wastewater is used and the importance of avoiding contact and health education program. The distance to residential areas and roads is for example set by law in Israel (Shelef et al., 1987).

2.3.2 Prevention of Damage to Crops and Soils

Guidelines for evaluation of water quality for irrigation emphasize the long-term influence of water quality on crop production and soil conditions. A number of different water quality guidelines relate to irrigated agriculture and crop response to water quality (National Technical Advisory Committee to the Secretary of the Interior, 1972; FAO/UNESCO, 1973; Shainberg an Oster, 1978; Shelef et al., 1987; Rhoades and Loveday, 1990). They all cover different problems, and use slightly different systems. In this study the FAO classification (Ayers and Westcot, 1985) of guidelines will be used, because of its completeness and clear categorizing.

The four categories that are used for evaluation are:

- salinity;
- water infiltration rate problems;
- toxicity;
- miscellaneous problems.

The various water quality problems mentioned above often occur in combination, but they are more easily understood and solved if each factor is considered individually. Complex problems may affect crop production more severely than a single problem (Ayers and Westcot, 1985).

Salinity

Salinity problems occur when salts in soil or water reduce water availability to the crop to such an extent that yield is affected. Salinity, measured by electrical conductivity (EC_w) or total dissolved solids (TDS), is the most important parameter in determining the suitability of water for irrigation (Ayers and Westcot, 1985). The effect of salt on crop growth is believed to be of osmotic nature: excess salts increase the energy that the plant must expend to acquire water from the soil and undertake the biochemical adjustments necessary to survive. This energy is wasted in the sense that it is diverted from the processes that lead to normal growth and yield (Rhoades and Loveday, 1990; Umali, 1993).

Plant growth is suppressed when a threshold concentration value of salinity is exceeded. Symptoms of salinity damage are barren spots, areas of stunted growth, discoloration, leaf burn along the margin or at the tip, necrosis and defoliation. However a reduction of yield may occur without any visible symptoms of salt injury (Shainberg and Oster, 1978). The threshold value, at which growth is suppressed, is a function of the type of crop, the stage of plant growth, the irrigation method and frequency, environmental, and pedologic factors (Umali, 1993). Plants respond to salinity in distinct manners: the salt tolerance as a function of soil salinity of most agricultural crops is known well enough to give general salt tolerance guidelines. These guidelines are presented in figure 4.

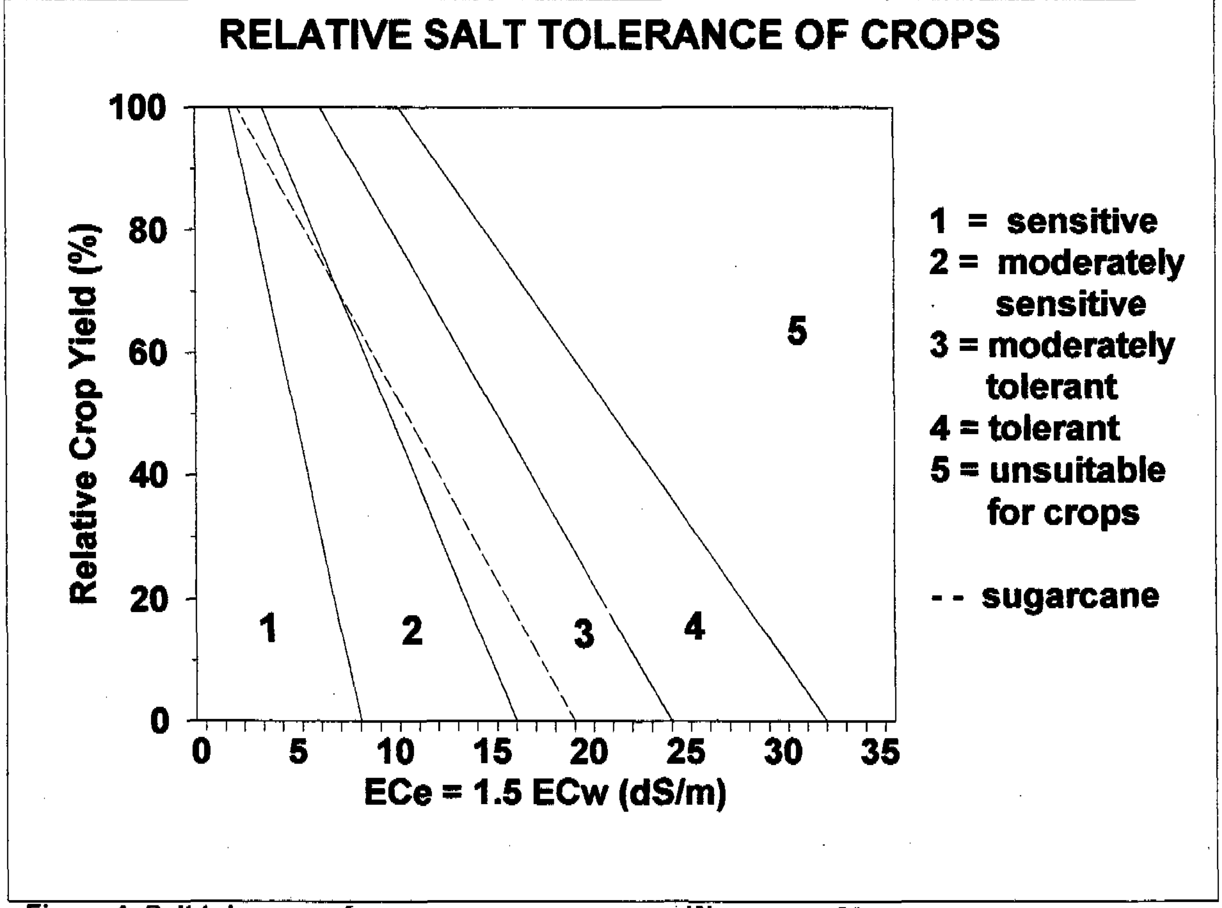


Figure 4: Salt tolerance of various crops (Ayers and Westcot, 1985)

Irrigation-induced salinity often is a man-made problem, caused by a complex mix of technical, political, social and economical reasons. At a technical level, irrigation-induce salinity has developed in some areas due to:

- inadequate leaching;
- poor construction, operation and maintenance of irrigation canals, leading to excessive seepage;
- the inadequacy or lack of drainage infrastructure;
- the poor quality of construction, operation and maintenance of drainage structures;

These technical problems, however, maybe the product of several other economical, political and social factors. Salinity can be caused by poor project planning and implementation; underpricing water; scarce financial resources of governments to undertake corrective measures; the short-term outlook and inadequate priority assigned to agricultural sustainability and environmental protection by policymakers; and the inability of donor agencies to ensure adherence to project plans (Umali 1993).

Irrigation-induced salinity occurs when the salt balance in the soil is disturbed, consequently salinity can be controlled by restoring the salt balance. Salts are added to the soil with each irrigation and removal by crops is mostly insufficient to maintain salt balance. Accumulation depends upon the quantity of salt applied in the irrigation water (salts in) and the rate at which salt is removed by leaching (salts out). The salt balance of a root zone can be written in terms of volumes which flow in and out of the rootzone (D [m^3]) concentrations (C [g/m^3]), and amounts of salts (S [g]). The net difference between input and output gives the resultant change in soil-water salinity or storage (ΔS_{sw}).

$$D_{iw}*C_{iw} + D_r*C_r + D_{gw}*C_{gw} + S_m + S_f = D_{dw}*C_{dw} + S_p + S_c + \Delta S_{sw} \qquad \textbf{(1)}$$

where: D_{iw} = flow of irrigation water [m^3];
C_{iw} = salt concentration of irrigation water [g/m^3];
D_{rw} = flow of rainwater [m^3];
C_{rw} = salt concentration of rain water [g/m^3];
D_{gw} = flow of groundwater [m^3];
C_{gw} = salt concentration of groundwater [g/m^3];
D_{dw} = flow of drainage water [m^3];
C_{dw} = salt concentration of drainage water [g/m^3];
S_m = Amount of salt dissolved from minerals in the soils [g];
S_f = Amount of salt added by agricultural chemicals, such as fertilizers [g];
S_p = Amount of salt percolating into the soil after irrigation [g];
S_c = Amount of salt removed through crops [g];
ΔS_{sw}= Storage of salts [g];

Salt storage in the rootzone should equal zero for good salinity control, and be negative for reclamation of saline soils. The salt balance equation reduces to equation 2, when the following assumptions can be made:

- Rainfall contains zero salts (C_r=0);
- There is no capillary rise from the groundwater (D_{gw}=0);
- The dissolution of minerals, and salts added by fertilizers is zero (S_m=S_f=0);
- Loss of soluble salt through percolation, and loss through crop uptake do not contribute appreciably (S_p=S_c=0);
- There is a uniform areal application of water in the field;
- There is no storage of salts (ΔS_{sw}=0);

Concentration can be substituted by electric conductivity (EC), since the electric conductivity of a water is a reliable index of its total solute concentration within practical limits (Rhoades and Loveday, 1990).

$$\frac{D_{dw}}{D_{iw}} = \frac{C_{iw}}{C_{dw}} = \frac{EC_{iw}}{EC_{dw}} = LF \qquad \textbf{(2)}$$

where: EC_{iw} = electric conductivity of irrigation water [dS/m];
EC_{dw} = electric conductivity of drainage water [dS/m];
LF = leaching fraction [-];

By varying the leaching fraction of applied water that is percolating through the root zone, it is possible to control the concentration of salts in the drainage water within certain limits and, hence, to control salinity in the rootzone, which is the intermediate between EC_{iw} and EC_{dw}. The best means of controlling soil and water salinity is the provision of efficient irrigation with adequate but minimum leaching and drainage management that maintains a downward net flux of water in the soil. Good salinity control is more than setting standards for irrigation water quality: it is a complex mixture of design and operation. The primary concerns in water management for salinity control are (Oster and Rhoades, 1985, Rhoades and Loveday, 1990):

- crop management;

- land and soil management;
- irrigation management;
- drainage management;
- monitoring.

crop management
Because crops and different varieties of the same crop vary considerably in their tolerance to salinity, crops can be selected that produce satisfactorily for the particular conditions of salinity in the root zone. When selecting a crop, it is important to consider the crop's salt tolerance during seedling development, as this is often the most sensitive growing stage. Plant density may be increased to compensate for smaller plant size that exists under saline conditions. This increases interception of the incoming energy of the sun and therefore, crop yield relative to normal conditions (Rhoades and Loveday, 1990).

land and soil management
Land management should be aimed at the most uniform water application to secure the best salinity control. Where irrigation is by flood or furrow methods, careful landgrading, such as is possible using laser-controlled earth-moving equipment, is desirable. Salt accumulation can be especially damaging to germination and seedling establishment when raised beds or ridges are used and "wet-up" by furrow irrigation, even when the average salt levels in the soil and irrigation water are moderately low (Rhoades and Loveday, 1990). Fertilization may increase salinity problems if fertilizer, manures and soil amendments are placed too close to the germinating seedling of grown plants. Care, therefore, should be taken in placement and timing of fertilization (Ayers and Westcot, 1985).

irrigation management
The prime requirements for irrigation management for salinity control are frequent irrigation, adequate leaching, water table depth control, a well-managed delivery system and an appropriate field application system (Oster and Rhoades, 1985; Rhoades and Loveday, 1990).

Seepage losses in the delivery system are often a major cause of the development of high water tables and excessive soil salinity in irrigated lands. In order to minimize seepage losses special attention should be paid to the construction of canals, flow-measuring devices and water delivery policy. Seepage may be reduced by compacting the canal floor and walls or by lining them with less permeable materials. In order to identify seepage losses and oversupply to farms, a net of effective flow-measuring devices at critical points needs to be installed. Ideally, water delivery should be on demand, because water delivery for a fixed period, or in fixed amounts, encourage overirrigation (Rhoades and Loveday, 1990).

In general, improvements in salinity control come from improvements made in on-farm irrigation efficiency, by providing the appropriate amount of water at the appropriate time with uniformity of application. The ideal irrigation scheme would provide water more or less continuously to the plant to match evapotranspiration losses and to keep water content within narrow limits commensurate with adequate aeration and adequate loss in deep percolation for leaching. Each irrigation method has certain advantages and disadvantages. Well-designed trickle systems are an excellent application system for salinity control and higher levels of salinity in the irrigation water can be tolerated with these systems than with other methods of irrigation (Shainberg and Oster, 1978, Rhoades and Loveday, 1990). A well-designed sprinkler system applies water with good uniformity and will result in an excellent overall irrigation and adequate and uniform leaching (Ayers and Westcot, 1985, Rhoades and Loveday, 1990). Flood and furrow irrigation can cause variable growth by uneven water distribution due to inadequate levelling and variations in soil properties, but good water and salinity control can be achieved, if designed and operated properly (Shainberg and Oster, 1978). In furrow systems use of closed conduits instead of open waterways for laterals, reducing furrow lengths, and surge irrigation improve performance. Subirrigation, in which the water table is maintained high enough so the capillary fringe and the root zone coincide, is not suitable over the long term with saline water

(Rhoades and Loveday, 1990). Periodic flooding, along with crop rotation, is recommended for salinity control for both furrow and trickle systems (Oster and Rhoades, 1985).

drainage management
Drainage, either artificial or natural, is an indispensable part of any irrigation area to remain viable in the long term. Without drainage, groundwaters eventually rise to levels that allow salts to accumulate in the soil and the root zone to become waterlogged. Water collected in drains can be reused for irrigation in lower fields, if necessary diluted with better-quality waters.

monitoring
The proper operation of a viable, permanently irrigated agriculture requires monitoring, providing periodic information on the levels and distributions of soil salinity within the root zones and fields of the irrigated area. Direct monitoring of root zone salinity is recommended to evaluate the effectiveness of various management programs (Rhoades and Loveday, 1990).

Water Infiltration Rate
An infiltration problem related to water quality occurs when the normal infiltration rate for the applied water or rainfall is appreciably reduced, and water infiltrates too slowly to supply the crop with sufficient water to maintain acceptable yields. The combination of salinity and high sodicity causes clay particles to swell, and reduces the normal infiltration rate. Suspended solids further reduce the water infiltration rate of an already slowly permeable soil (Ayers and Westcot 1985).

Sodicity
The infiltration rate generally increases with increasing salinity and decreases with either decreasing salinity or increasing relative sodium content. Salinity and sodium adsorption ratio (R_{Na}) have to be considered together for a proper evaluation of the ultimate effect on water infiltration rate. If the soil has a low chloride and calcium content and if the soil or irrigation water applied have abundant exchangeable sodium bicarbonate and/or sodium carbonate, the clay particles in the soil adsorb the sodium and magnesium and swell. Through this mechanism the soil loses its permeability (ability to conduct air and water) and tilth (friability of the seedbed) (Umali, 1993).

Over the past few years several adjustments have been made to the calculation method of the sodium adsorption ratio. The method used here is the one presented by Ayers and Westcot (1985), which takes into account the effects of bicarbonate and salinity upon the calcium concentration, by using a modified calcium value.

$$adj\ R_{Na} = \frac{Na}{\sqrt{\frac{Ca_x + Mg}{2}}} \qquad \mathbf{(3)}$$

where: Adj R_{Na} =adjusted sodium adsorption ratio [-];
Na = sodium concentration [me/l];
Ca_x = modified calcium value [me/l];
Mg = magnesium concentration [me/l];

Management steps available to help maintain yields can be chemical or physical, as well irrigation management measures. Certain chemical amendments, such as gypsum, added to soil or water would improve a low infiltration rate caused by low salinity and high sodicity, by increasing the soluble calcium content or increasing electric conductivity of the applied water. By blending water supplies the sodicity and salinity of the irrigation water can be altered. Physical methods keep the soil open by mechanical means. These methods include cultivation and deep tillage. Crop residues or other organic matter left in the field improve water penetration and is becoming a more widely accepted practice, as it is one of the easiest measures to improve water infiltration (Ayers and Westcot, 1985).

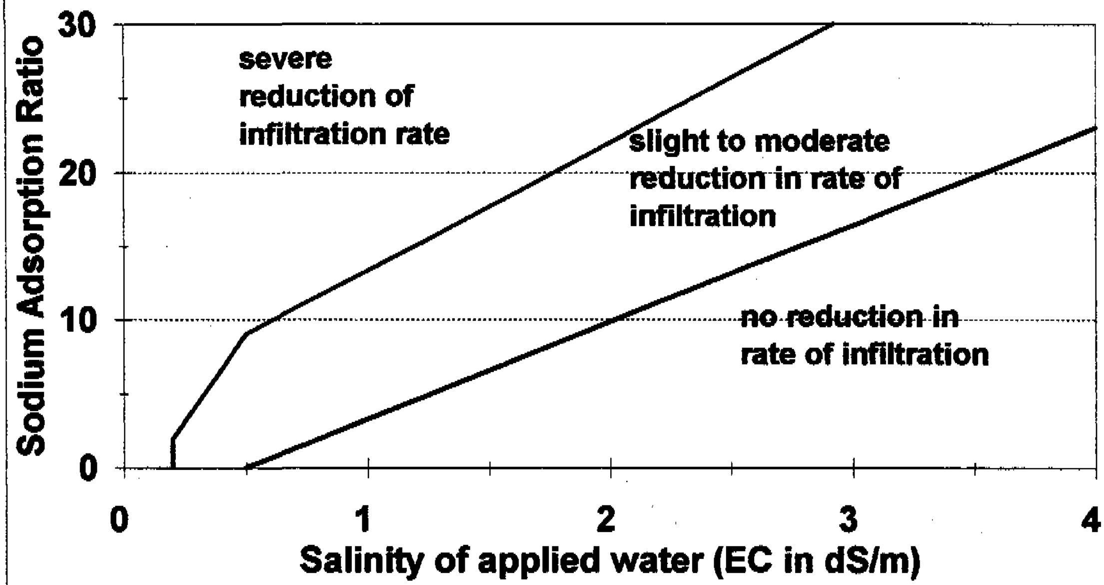


Figure 5: Relative rate of water infiltration as affected by salinity and SAR (Ayers and Westcot, 1985)

Irrigation management measures are less effective but also less costly than chemical and physical methods to control water infiltration problems. Frequent irrigation is a simple and effective approach especially for soils having an initially high infiltration rate but for which the rate drops rather quickly due to low salinity or high sodicity. Preplant irrigation can be relied upon to fill the rooting depth to field capacity at a time when there is little change of causing crop damage. Extending the duration of an irrigation applies more water and is beneficial provided drainage is adequate. Changing the irrigation system to localized irrigation may allow the user to approach the soil intake rate more closely (Ayers and Westcot, 1985).

Solids in irrigation water can further reduce already low infiltration characteristics of slowly permeable soils. The effect depends on the particle-size distribution of the suspended material. Deposition of colloidal particles on the soil surface can produce crusts which inhibit water infiltration and seedling emergence. This same deposition and crusting can reduce soil aeration to a level where it impedes plant development (National Technical Advisory Committee, 1972).

Although the role of magnesium in causing or partly causing soil infiltration problems is not well documented, it can be said that a given sodium adsorption ratio will show slightly more damage if the water is magnesium dominated (ration of Ca/Mg < 1). There are however insufficient data to make the Ca/Mg ratio an evaluation factor when judging the suitability of a water for irrigation (Ayers and Westcot, 1985).

Toxicity
Toxicity problems occur if certain ions in the soil or water are taken up by the plant and accumulate to concentrations high enough to cause crop damage or yield reduction. A toxicity problem occurs within the plant itself and is not caused by a water shortage. The degree of damage depends on the uptake and the crop sensitivity. Permanent crops are more sensitive than annual crops, because of possible accumulation of toxic ions.

The substances of primary concern are (1) specific ions, (2) trace elements, and (3) trace organics. The specific ions boron, chloride and sodium can be toxic by plant uptake and by absorption of leaves when overhead sprinkler irrigation is applied, both processes causing leaf burn. Specific ion toxicity is commonly associated with woody species and rarely occurs among herbaceous plants (Shainberg and Oster, 1978). Trace elements are toxic to plants at very low concentrations, but most irrigation supplies contain very low concentrations of these trace elements. Wastewater however, especially from industrial sources, could contain harmful concentrations (Ayers and Westcot, 1985). Trace organics can cause environmental risks.

Specific ions
Boron is a constituent of practically all natural waters. It is essential for plant growth but is exceedingly toxic at concentrations only slightly above optimum. Symptoms of boron toxicity include yellowing, spotting and drying of leaves. Sensitivity to boron varies widely, however, prolonged use of water containing boron levels exceeding 3 mg/l is not generally recommended (Shainberg and Oster, 1978). Boron is not removed effectively during wastewater treatment (Page and Chang, 1985a).

Chloride is not adsorbed or held back by the soil and therefore is taken up by plants and accumulates in the leaves. If the chloride concentration in the leaves exceeds the tolerance of the crop, injury symptoms develop, such as leaf burn and drying of leaf tissue. Chloride tolerances vary: woody crops and fruit crops are the more sensitive. (Ayers and Westcot, 1985; Rhoades and Loveday, 1990). Oster and Rhoades (1985) state that chloride is not toxic to vegetable, grain, forage or fiber crops.

Sodium not only affects soil permeability, but can also have a direct toxic effect. Typical toxicity symptoms are stunted growth, leaf burn, scorch and dead tissue along the outside edges of the leaves (Shainberg and Oster, 1978, Ayers and Westcot, 1985). Direct toxicity effects related to sodium are generally limited to perennial woody species; injury is common in avocado, citrus, nuts, beans and stone-fruit trees (Ayers and Westcot, 1985; Oster and Rhoades, 1985; Rhoades and Loveday, 1990). Sodium toxicity is often modified or reduced if sufficient calcium is available in the soil. The measure used for sodium toxicity is the exchangeable sodium percentage (ESP) of the of the soil. Sugarcane is semi-tolerant to exchangeable sodium, its tolerance being 14-40 ESP, which is normally reached as the sodium adsorption ratio of irrigation water exceeds 13 me/l (Ayers and Westcot, 1985).

Toxicity from boron, chloride and sodium can be controlled using cultural practices. Potentially toxic ions can be reduced by leaching in a manner similar to that of salinity, but the depth of water required varies with the toxic ion and may in some cases become excessive. Increasing the frequency of irrigation supplies a greater proportion of the water needs from the upper soil, where toxic ion concentration are generally lower. Toxicity problems can also be solved by stimulating vegetative growth by extra fertilization. Sodium toxicity from applied water can be countered by use of a soil or water amendment such as gypsum. If these cultural practices can not counteract toxicity, either the water supply should be blended or changed or the crop grown should be changed (Ayers and Westcot, 1985).

A chloride or sodium toxicity can also occur by direct leaf absorption through leaves wetted during overhead sprinkler irrigation (foliar adsorption). Absorption and toxicity occur mostly during periods of high temperature and low humidity (<30 percent), frequently aggravated by windy conditions. Rotating sprinklers heads present the greatest risk, because water evaporates between rotations. High frequency (near daily) spray irrigation can also create problems. Crop tolerances to sodium and chloride in sprinkler-applied irrigation are not well established and depending on climatic conditions (Ayers and Westcot, 1985). Foliar adsorption is negligible in sugarcane cultivation (Shainberg and Oster, 1978).

Trace elements
In small quantities many elements are essential to biochemical growth, while others have no physiological function; at a slightly higher concentration, many elements may become toxic to plants. The term trace element is used to denote a group of otherwise unrelated chemical elements present in the natural environment in low concentrations. In the soil uncontrolled trace element inputs are undesirable, because once accumulated in the soil, these substances are in most cases practically impossible to remove and may lead to toxicity to plants grown, exposure of humans through crops, and transport to ground or surface water, thus making this water unfit for its intended use (Page and Chang, 1985a).

Trace elements are effectively removed from wastewater by removal of suspended solids. Among the trace elements commonly found in municipal wastewater, cadmium, copper, nickel and zinc are considered to present a potentially serious hazard if they are introduced into the cropland in an uncontrolled manner. Following common crop production practices, manganese, iron, aluminum, chromium, arsenic, selenium, and lead inputs through application of treated domestic wastewater to land should not result in toxicity or expose humans to potentially hazardous trace elements levels (Page and Chang, 1985a).

Since trace elements concentrations of wastewater vary considerably, it is essential that cropland irrigation operations should be evaluated case by case. Various organizations have made systems of guidelines for irrigation water quality regarding trace elements. These threshold limits are intended to protect even the most sensitive plants from harmful effects. Page and Chang (1985a) state that water containing higher concentrations may be suitable for irrigation if their use is carefully planned and managed. A review of existing guidelines is given in table 5, appendix VII.

Trace organics
Trace organics are those organic substances that are present in seemingly uncontamined water or treated wastewater effluents in extremely low concentrations. As they are only discovered quite recently, data on these substances are scarce. Conventional wastewater treatment processes greatly reduce the number and concentrations of trace organics and the environmental risk should not be greater than that associated with using surface water, which also contains trace organics. Environmental impacts associated with the application of pesticides outrank these of the use of reclaimed wastewater (Page and Chang, 1985b).

Miscellaneous
Three other parameters cause damage to crops and soils besides salinity, reduction of water inflitration rate, and toxicity: high nitrogen concentrations, high BOD-concentrations, and scale formation. High nitrogen concentrations can cause excessive vegetative growth and delayed crop maturity. High BOD-concentrations indicate high biodegradable organics concentrations, which can cause oxygen deficits in the soil. White scale formation on fruit or leaves can result from overhead sprinkler irrigation with high bicarbonate water, water containing gypsum, or water high in iron.

Nitrogen is a plant nutrient and stimulates crop growth. Natural soil nitrogen or added fertilizers are the usual sources, but nitrogen in the irrigation water has much the same effect as soil-applied fertilizer nitrogen. Excess nitrogen in irrigation water causes problems, just as too much fertilizer. If excessive quantities are present or applied, production of several commonly grown crops may be upset because of over-stimulation of growth, delayed maturity or poor quality. Total nitrogen concentrations of less than 5 mg/l has little effect, even on nitrogen sensitive crops. Sensitive crops may be affected by total nitrogen concentrations above 5 mg/l; most other crops are relatively unaffected until total nitrogen exceeds 30 mg/l. Fertilizer requirements of sugarcane are high, and sugarcane can be regarded as tolerant to high nitrogen levels (Jones et al., 1990). The sensitivity of the crop varies with the growing stage. High nitrogen levels are beneficial during early growth stages, but may cause yield losses during the later flowering and fruiting stages. Blending or changing supplies during the later more critical growth stages should be helpful. For crops irrigated with water containing nitrogen, the rates of nitrogen fertilizer supplied to the crop can be reduced by an amount very nearly equal to that available from the water supply (Ayers and Westcot, 1985).

Nitrogen in wastewater applied to land is subject to leaching if not intercepted by plants roots, immobilized by microorganisms, or denitrified. Eutrophication, the accumulation of nitrogen in surface water and groundwater can cause algal bloom and subsequent O_2 depletion in surface water. It may result in fish kills, and a decrease in esthetic and recreational value. Nitrates may accumulate in groundwater and present a public health problem, especially to infants younger than 12 weeks of age (Hornsby, 1990). However, the contribution of surface-applied nitrogen to this accumulation is not well defined. Estimates of the quantity of nitrogen leached in a given situation can be made by subtracting nitrate utilized by the crop from the total nitrogen applied and then using a reasonable estimate of denitrification loss to adjust the remainder (Broadbent and Reisenauer, 1985).

The significance of BOD-values of irrigation water has not been fully assessed, though soil aeration and oxygen availability are reckoned to be a factor deterring plant growth. BOD is not likely to be a problem were sprinkler irrigation, which provides considerable aeration, is used and adequate drainage is provided. The National Technical Advisory Committee (1972) does not prescribe specific criteria, because of insufficient information. Shelef et al. (1987) quote Israeli limits varying from 25 to 60 mg/l depending on the crop. Arthur (1983) and Culp and Hinrichs (1981) suggest a maximum BOD-concentration of 35 mg/l for irrigation, the US Department of the Interior (1966) suggests an even lower limit of 20 mg/l, other sources do not set BOD-limits.

Irrigation water containing a high proportion of slightly soluble salts such as calcium, bicarbonate and sulphate presents a problem of white scale formation on leaves or fruit when sprinklers are used. Calcium deposits form even at very low concentrations if sprinklers are used during periods of very low humidity (< 30 percent), resulting in a high rate of evapotranspiration. Management options to avoid deposit problems included adding acid material to high calcium water, irrigation at night, increase of the speed of sprinkler rotation, and decrease of irrigation frequency (Ayers and Westcot, 1985).

2.3.3 Prevention of Nuisance Conditions during Storage and Operation

Poor quality irrigation water can create problems in operation and maintenance of the irrigation system. The major problems that can occur are: water-induced corrosion, clogging of equipment by suspended solids, and nuisance growth of algae and plants.

Due to water-induced corrosion or encrustation deterioration of equipment can occur. This problem is most serious for wells and pumps, but a poor quality water can also damage irrigation equipment and canals. Low pH water is very corrosive and may rapidly corrode pipelines, sprinklers and control equipment (Ayers and Westcot, 1985).

Suspended solids cause problems in irrigation systems through clogging of gates, sprinkler heads and drippers and sedimentation of canals and ditches. This can cause costly dredging and maintenance problems (Ayers and Westcot, 1985). On the other hand suspended solids are fertile particles that are beneficial to plant growth, and care should be taken to transport fertile solids to the irrigated fields. Small particles of suspended solids can penetrate the canal walls and bottom and thus decrease permeability and seepage (auto-colmatage) (FAO/UNESCO, 1973). Where sprinkler irrigation is used deposition on leaf surfaces could occur, which could reduce photosynthetic activity and therefore deter growth (National Technical Advisory Committee, 1972). Ayers and Westcot (1985) suggest a TSS limit of 50 mg/l for trickle irrigation systems, and more flexible limits for sprinkler and gravity systems. Culp and Hinrichs (1981) quote a value of 50 mg/l, regardless of the distribution system. Shelef et al. (1987) quote Israeli maxima of 20 to 50 mg/l, depending on the crop (see table 49, appendix VII). Telsch et al. (1991) stress the importance of clogging capacity of reclaimed wastewater as a quality criteria for drip irrigation, but propose other parameters, such as filterability tests for evaluation of water quality.

Nitrogen may stimulate nuisance growth of algae and plants, which clogs equipment, decreases channel capacity and increases maintenance costs. Very low nitrogen concentrations (< 5 mg/l) can cause nuisance growth, especially when temperature, sunlight, and other nutrients are optimum. Nitrogen problems can either be managed by mechanical controls such as screens and filters or chemical control with materials such as copper sulphate. (Ayers and Westcot, 1985). The National Technical Advisory Committee (1972) states a maximal nitrogen concentration of 45 mg/l.

2.4 Macrophyte-Based Treatment Systems

Macrophyte-based are natural wastewater treatment systems in which aquatic macrophytes (wetlands plants, which are adapted to growing in water saturated soils) have a key function in relation to the cleaning of wastewater (Brix, 1993). This paragraph discusses the differences between natural and mechanized systems (§2.4.1); the definition of macrophyte-based treatment systems (§2.4.2), four examples of macrophyte-based treatment system (§2.4.3-§2.4.6), and a comparison of these systems (§2.4.7).

2.4.1 Natural versus Mechanized Treatment

Biotechnical systems for wastewater treatment can roughly be grouped into two categories: (1) mechanized systems and (2) natural wastewater systems. All biotechnical systems depend on natural responses such as gravity forces for sedimentation, or on natural components such as biological organisms; natural treatment systems are systems that depend primarily on their natural components to achieve the intended purpose of treating wastewater.

Natural treatment systems offer effective and reliable wastewater treatment and have several advantages and disadvantages when compared to mechanized systems. Natural systems are relatively inexpensive to construct and operate and easy to maintain. They are relatively tolerant of fluctuating hydrological and contaminant loading rates and may provide indirect benefits such as green space, wildlife habitats, and recreational and educational areas. In general natural systems are favored in tropical climates: due to the higher temperature of the sewage, most bioconversion processes are accelerated which may result in cost savings. Furthermore in tropical countries land and unskilled labor are often comparatively cheaper than equipment. Land prices largely determine investments in natural treatment systems. Arthur (1983) estimated the break-even point, above which the choice of a mechanized system becomes preferable, at about US$8 per square meter. Disadvantages of natural systems for wastewater treatment compared to conventional systems include: relatively large land area requirements for advanced treatment; current imprecise design and operating criteria; biological and hydrological complexity; lack of understanding of important process dynamics; and possible problems with pests. In moderate climates natural technologies are often not advocated as their performance is very low at winter periods and land costs are comparatively high (Hammer and Bastian, 1989; Al-Nozaily, 1995).

2.4.2 Natural treatment: Macrophyte-based Systems

Natural wastewater treatment technology is emerging as a low-cost, easily operated efficient alternative to conventional treatment systems for a wide variety of wastewaters (Watson et al., 1989). There are several systems of categorizing natural treatment systems: Reed et al. divide natural systems in three major categories: aquatic, terrestrial, and wetlands concepts; others classify systems according to the life form present in the system (Pescod and Mara, 1988; Brix, 1993). Natural treatment systems can also be divided hydrologically, into discharging systems (with an outfall or other direct discharge) and nondischarging systems (Reed et al., 1995). I differentiate according to the life form between pond systems with floating macrophytes (hyacinth ponds and duckweed ponds), and wetlands with rooted macrophytes (free water surface wetlands and subsurface flow wetlands).

Wastewater stabilization ponds have been in operation for many years. One of their drawbacks is, however, the presence of algae in the effluent. This reduces treatment efficiencies of suspended solids, and pond effluent therefore is sometimes not suitable for use in irrigation. Recently other natural treatment concepts, which produce effluent with no algae present, have been put into practice, especially macrophyte-based systems, such as hyacinth ponds, duckweed ponds and constructed wetlands. Information on these systems is limited and sometimes inconsistent: their greatest drawback is the lack of detailed information from long-term experience (Pescod and Mara, 1988; Hammer and Bastian, 1989; Sapkota and Bavor, 1994).

Macrophyte-based wastewater treatment systems are defined as natural wastewater treatment systems in which aquatic macrophytes (wetlands plants, which are adapted to growing in water saturated soils) have a key function in relation to the cleaning of wastewater (Brix, 1993). Macrophyte-based treatment systems removed pollutants by a complex variety of biological, chemical and physical processes, including sedimentation, filtration, soil adsorption, microbiological degradation, (de)nitrification, natural die-off and plant uptake. Macrophytes remove pollutants by assimilating them into their tissue and by providing surfaces and suitable environment for microorganisms to transform pollutants. Oxygen transfer into the rootzone contributes to pollutant removal, by creating an aerobic environment for certain bacteria. Most macrophyte-based systems serve two purposes: wastewater treatment and the production of biomass. Typical biomass crops yielded include: reeds for thatching, building and matting, wood for building, plants for animal feeding, and biomass for production of biogas. Some natural treatment systems serve as a wildlife habitat (Hillman and Culley, 1978; Pescod and Mara, 1988). It is not clear if natural systems can be optimized for both waste treatment and protein production at the same time (Reed et al., 1995).

Macrophyte-based treatment systems may be classified according to the life form of the dominating macrophyte into (Brix, 1993):

- Free-floating macrophyte-based treatment systems: hyacinth and duckweed ponds;
- Rooted emergent macrophyte-based wastewater treatment systems: free water surface and subsurface flow wetlands;
- Submerged macrophyte-based wastewater treatment systems;

The latter type, submerged macrophyte-based systems is not discussed here. Submerged systems are ponds with small plants which are entirely submerged; these plants are not considered a useful crop and harvesting is only carried out for maintenance purposes (Brix, 1993). This concept therefore does not meet the criteria set for the pre-irrigation treatment to be designed.

Free floating macrophyte-based systems are aquatic wastewater treatment systems. Some sources look upon them as upgraded wastewater stabilization ponds (Pescod and Mara, 1988), others as a kind of wetlands (Brix, 1993), others as a separate kind of treatment concept (Reed et al., 1995). A free floating macrophyte-based wastewater treatment system typically consists of an anaerobic pond, a maturation pond, and one or more maturation ponds covered with free floating macrophytes. No design for floating macrophyte ponds has been sufficiently evaluated to confirm its long term efficiency in operation, but waste stabilization pond design concepts are used for the largest part. Free floating macrophytes are highly diverse in form and habit, ranging from large plants with rosettes of floating leaves and well-developed submerged roots, to minute surface-floating plants with few or no roots. Hyacinths, pennyworth and duckweeds are the only varieties tested in pilot or full-scale systems to date; hyacinth and duckweeds are discussed here, as they are examples of both extremes of free floating macrophytes.

Wetlands are, as the word indicates, wet lands: areas flooded or saturated often and long enough to support those types of vegetation and aquatic life that require or are specially adapted for saturated soil conditions (Hammer and Bastian, 1989). They represent a transition between terrestrial and aquatic systems, and contain features of both. Natural wetlands can be used for wastewater treatment, but can be severely damaged by the pollutants in the influent and are difficult to control. Most wetlands presently in use for wastewater treatment are constructed specially for wastewater treatment (Brix, 1993; Osborne and Totome, 1994). A constructed wetland (also referred to as man-made, engineered and artificial wetlands) is a designed and man-made complex of saturated media, emergent and submergent vegetation, animal life and water. It simulates natural wetlands for human use and benefits (Hammer and Bastian, 1989). Municipal wastewater treatment systems using constructed wetlands may be categorized hydrologically as either free water surface types (also referred to as surface systems) or subsurface flow types (also referred to as rootzone systems, underground wetlands, gravel beds, rock beds or rock-reed-filters). Both types are discussed here.

Recently some vertical subsurface flow systems have been put into operation. The sparse information on their treatment performance indicates good performance with respect to TSS, BOD, nitrogen and phosphorus. They require up to 50% less space. The few systems in operation are located in moderate climates and even in Europe full scale design and operating is not yet recommended, because of the lack of operating data (Brix, 1993; Brix 1994; Cooper, 1993). Vertical flow beds are not considered here.

2.4.3 Hyacinths Ponds

The most common floating plant used in aquatic wastewater systems is the water hyacinth *(Eichhornia crassipis)*. The water hyacinth is one of the fastest growing plants in the world, is resistant to many insects and diseases, and can thrive in raw sewage. Often water hyacinth are regarded as a weed, but their high productivity is exploited in wastewater treatment facilities.

Removal of BOD is good, 80-90%, and is largely performed in the anaerobic and the facultative pond. TSS removal is good, 70-90%. Data on nitrogen vary from low to high (10-90%). Phosphorus removal is moderate (15-60%), but can be increased by frequent harvesting. In a survey of macrophytes, hyacinths showed the highest levels of nitrogen removal, and moderate results on phosphorus removal (Reddy and DeBusk, 1985). Heavy metals removal is excellent, over 90% in the anaerobic pond and more in the hyacinth maturation ponds (Mara and Mills, 1994). Pathogen removal is said to be over 90% (Vroon and Weller, 1995; Reed et al., 1995; Reed, 1990). Helminth eggs removal is nearly complete, due to long retention times.

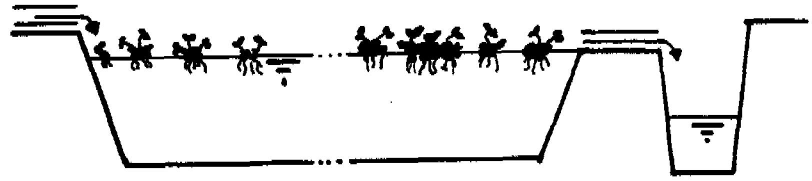


Figure 6: hyacinth pond

Water hyacinth are used in pond systems with total retention times varying normally between 5 and 20 days and depths between 0.5 and 2 meter(Vroon and Weller, 1995). The evapotranspiration from the surface of a hyacinth pond equals open water surface evaporation (Reed et al., 1995). Harvesting of hyacinth biomass is often only carried out for maintenance purposes, as its value is low. Plants can be used however as fertilizers, for biogas production, fuel, and hardboard production (Philipp et al., 1983; Edwards, 1992). Plants cannot be used as fodder, because of their low protein content.

2.4.4 Duckweed Ponds

Duckweeds have been investigated less than water hyacinths for use in wastewater treatment, but have shown potential usefulness in the treatment of eutrophatic water systems. Uses of duckweeds include BOD and TSS removal and recovering nutrients from secondary treated wastewater. Duckweed is cold tolerant and less sensitive than other aquatic plants to high nutrient stress, droughts, pests and diseases (Koles et al., 1987). Duckweed-based systems may face problems of high winds piling the duckweed, however this can be prevented by the construction of barriers on the water surface.

Data on the removal of BOD vary largely: Al-Nozaily (1995) cites results varying between 50-70%; Reed et al. (1995) state BOD removal is good, Vroon and Weller (1995) cite data between 20 and 96% in duckweed ponds. An estimate of 80% removal in the complete system, including anaerobic and facultative pond, is prudent. TSS removal is excellent (90-95%), mostly because of long retention time allowing settling by gravity (Vroon and Weller, 1995; Reed et al., 1995). Data on nutrient removal are numerous but inconsistent. Nitrogen and phosphorus removal are claimed to be significant with frequent harvest and long retention times (Koles et al., 1987; Whitehead et al, 1987; Reed, 1990; Vroon and Weller, 1995); nitrogen removal is claimed to be better than phosphorus removal (Reed et al, 1995); duckweed nitrogen removal potential was the lowest in a survey of macrophytes (Reddy and DeBusk, 1985). Heavy metal removal is good, especially in anaerobic ponds, where over 90% of the heavy metals are removed (Mara and Mills, 1994; Reed et al., 1995). Helminth eggs removal is almost complete for ponds with detention times of over 10 days (Mara et al., 1992; Vroon and Weller, 1995; El Nozaily, 1995).

Retention times of duckweed pond systems vary normally between 2 and 30 days (Whitehead et al, 1987; Vroon and Weller, 1995). Depth varies between 0.3 - 2 meter, optimal depth is subject to discussion (Brix, 1993; Vroon and Weller, 1995). Evapotranspiration is equivalent to open water surface evaporation and is very high due to the large surface area. (Reed et al., 1995). The nutritive, and therefore the economic, value of duckweed is high compared tot that of water hyacinth: harvested duckweed may be used as animal feed (for fish, cattle and poultry) or compost for land application. Duckweed is easily harvested from the water surface (Hillman and Culley, 1978; Reed et al., 1995).

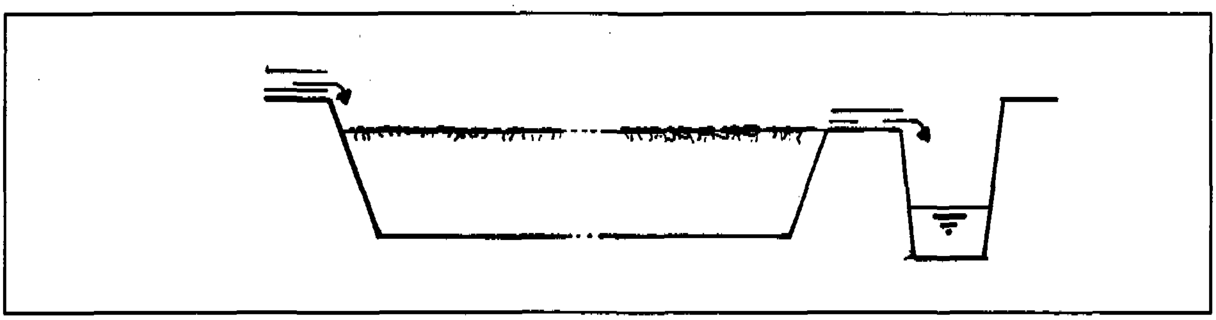


Figure 7: Duckweed pond

2.4.5 Free Water Surface Wetlands

Free water surface wetlands consist of basins or channels with a natural or constructed subsurface barrier to prevent seepage. Emergent vegetation is grown in soil or other suitable media. Wastewater is treated as it flows shallowly through the vegetation and plant litter. Removal of pollutants depends on interaction between air, plants and water (Brown, 1994). Channels are typically long and narrow, ensuring approximate plugflow conditions and minimizing short-circuiting (Watson et al.,1989; Crites, 1994).Water depths can range from several centimeters to 0.8 m, a typical operating depth is 0.3 m. The largest free water surface system is a system is under design in Egypt with a design flow of $1 * 10^6$ m^3/d (Reed et al., 1995).

Consistently good BOD and TSS removal (>60%) can be expected from free water surface wetlands (Reed et al., 1995). Effluent concentrations are estimated less than 20mg/l for BOD and less than 10 mg/l for TSS (Brown, 1994; Reed, 1990). Nutrient removal is more difficult to predict: Reed (1990) states total nitrogen effluent concentrations of less than 10 mg TN/l, Brown (1994) mentions varying nitrogen removal due to oxygen deficiencies in terms of nitrification requirements. Phosphorus removal varies widely between 20 and 50% (Reed et al., 1995) and appears to be completely dependent on the type of media or soil and so performance is very site specific (Brown, 1994). Metals removal is excellent, removed metals include lead, zinc, cadmium, iron, cobalt, nickel, copper manganese, and uranium (Noller et al., 1994; Reed et al., 1995). Pathogen removal is good, but extra pathogen input by animals is possible (Reed et al., 1995), helminth eggs removal is good due to the filtration through the stems of the macrophytes.

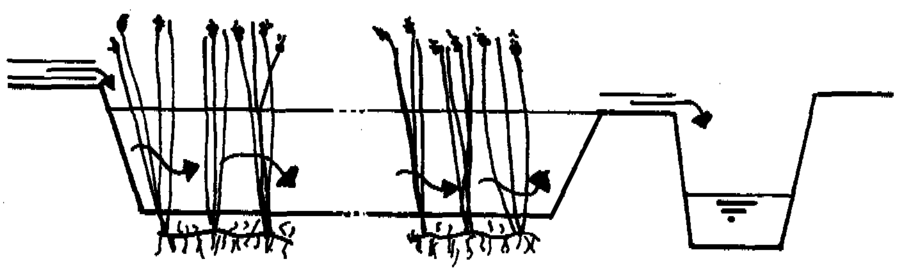


Figure 8: Free water surface wetland

Free water surface wetlands occupy very large areas of land: evaporation is very high as it is a function of the surface area: it is estimated at 0.8 times open water surface evaporation (Kadlec, 1989; Reed et al., 1995). Typical harvest consist of reeds, which may be used as fuel, handicraft or construction material. Free water surface wetlands offer a high habitat value for wildlife. They can, however, create several public health hazards, such as mosquito problems, odors, and danger for coypu and muskrats. Problems can be partly resolved by decreasing public access, proximity to homes and vector control (Brown, 1994).

2.4.6 Subsurface Flow Wetlands

Subsurface flow wetlands consist of a trench or bed containing media, such as crushed stones, gravel, and different soils, that supports growth of emergent vegetation. Water flows horizontally through the rootzone of the wetland plants, treated effluent is collected in an outlet channel or pipe (see figure 9). Removal of pollutants depend largely on interaction between media and water (Brown, 1994). Beds slopes typically are less than 2% and are underlain by impermeable material to prevent seepage and assure water level control. The depth of the media is typically 0.3 to 0.7 meter. The largest system in use in USA is in Crowley, Louisiana, with a design flow of 13,000 m^3/day (Reed et al., 1995).

Subsurface flow wetlands have been investigated less than free water surface wetlands and performance data vary widely. High BOD and TSS removal efficiencies (60%- 90%) can be expected from subsurface flow wetlands (Cooper, 1993; Reed et al., 1995). Effluent concentrations of systems with hydraulic detention time greater than 2 days are less than 20 mg/l for BOD and 5-15 mg/l for TSS (Brown, 1994, Reed, 1990). Nitrogen removal is up to 75%, however, many wetlands systems appear to be oxygen deficient in terms of nitrification requirements (Watson et al, 1994). Phosphorus removal is dependent on the type of media or soil and so performance, varying between 30-50%, is very site specific (Brown, 1994, Cooper, 1993; Reed et al., 1995). Total phosphorus effluent concentrations vary between 3 and 10 mg/l (Reed, 1990). Trace elements removal is said to be excellent, as both aerobic and anaerobic conditions exist (Eger, 1994; Reed et al., 1995). Pathogen removal efficiency is higher than 90% (Reed et al., 1995), helminth eggs removal is nearly complete.

Loading rates of subsurface flow wetlands are considerably higher than those of other macrophyte based treatment concepts, consequently the surface area required is considerably smaller. Evaporation is high, it is estimated to be 0.8 times evaporation of open water surfaces, but other methods of calculating are used as well (Kadlec, 1989; Reed et al., 1995). Harvested products typically include wood and reed which may be used as fuel, handicraft or construction material. Clogging problems with subsurface flow systems (Brown, 1994) can cause surface flow and low removal of nitrogen. Since the water surface is not exposed, there are no public-access and mosquito problems.

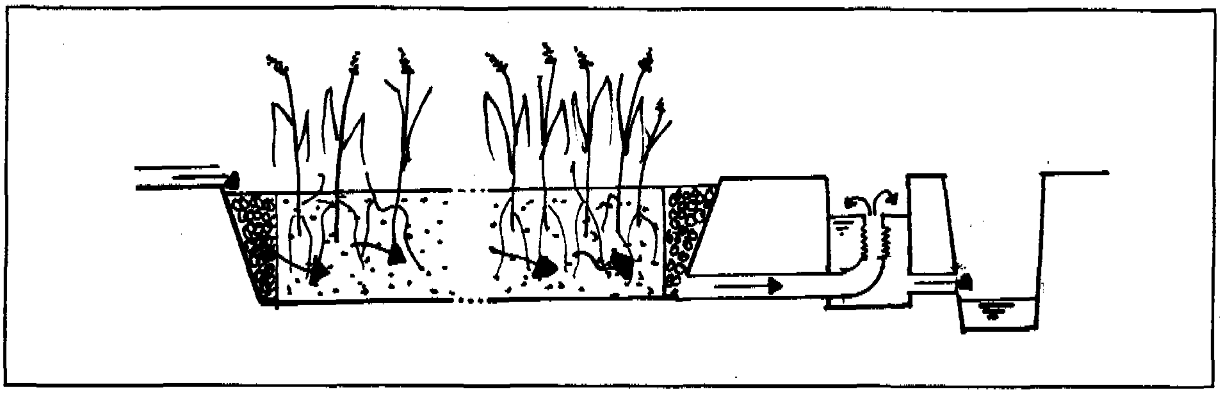


Figure 9: Subsurface flow wetland

2.4.7 Comparison of Macrophyte-Based Treatment Systems

Suitability of a certain system for a particular project should be primarily based on treatment requirements. Only concepts that can satisfy the pollutant removal requirements should be included in further evaluation, based on the other differences, presented in table 2.

The surface area occupied by a system is an important parameter when comparing treatment concepts, especially in areas with fairly high ground prices, like the Valle de Cauca. Estimates of required area vary widely in literature. Although the surface area of a treatment depends on the removal of several pollutants, including BOD, TSS, nutrients, pathogens, and in the case of wetlands hydraulic loading. A preliminary comparison of surface areas, however, can be based on BOD-loading rates of the treatment concepts. Hyacinth ponds and duckweed ponds both have a maximum loading rate of about 50 kg BOD/ha/day (Reed, 1995). This is considerably lower than conventional facultative ponds, because they are very shallow. Free water surface wetlands have a loading rate of 80- 200 kg BOD/ha/day (Reed et al, 1995), the maximum loading rate in operation being 54 kg BOD/ha/day (Brown and Reed, 1994). Subsurface flow wetlands have a loading rate of 75 to 600 kg BOD/ha/day, the maximum loading rate to be in operation in the USA is 155 kg BOD/ha/day (Brown and Reed, 1994). A higher BOD-loading rate results in a smaller surface area. Based on these data the relation between surface areas is:

hyacinth pond : duckweed pond : free water surface wetland : subsurface flow wetland = 3 : 3 : 3 : 1

All macrophyte-based wastewater treatment systems are cheap to construct, to maintain and to operate, when compared to conventional systems. Some cost estimates do exist, but they are hard to compare as all of them do include different parts of the treatment system, and are made in various years and countries. The actual construction and operation cost will depend largely on local circumstances.

Construction costs of hyacinth ponds, duckweed ponds and free water surface wetlands per m^2 are comparable, while costs of subsurface flow wetlands are higher, as they involve the purchase of vast volumes of gravel media. Construction costs of pond systems in the USA are reported to be 500-1000 US$ per m^3/day (Reed 1990). Construction costs of wetland systems in the USA are reported to be in the range of US$ 20-2000 per m^3/day (Reed 1990; Brown, 1994; Brown and Reed, 1994). Some sources state that construction costs of subsurface flow wetlands per m^2 are up to 50% higher than those of free water surface wetlands because of the cost of placing of medium.

Table 2: Comparison of macrophyte-based treatment concepts

	Hyacinth ponds	**Duckweed ponds**	**Free water surface wetlands**	**Subsurface flow wetlands**
Removal				
BOD	good	moderate to good	good	good
TSS	good	excellent	good	excellent
Nitrogen	moderate to good	poor to good	moderate to good	moderate to good
Phosphorus	moderate	inconsistent data	inconsistent data	inconsistent data
Coliform	excellent	moderate	good	good
Helminth eggs	excellent	excellent	good	excellent
Trace elements	excellent	good	excellent; limited data	excellent; limited data
Tropical experience	moderate	moderate	scarce	scarce
Area of land	very large	very large	very large	large
Temperature	desirable 20-30 °C; survival >10 °C	optimum 20-30 °C; survival 5–33°C	the higher the better	the higher the better
Construction costs	very low	very low	very low	low
Operation costs	low	low	low	low
sludge removal	every 2-3 years	every 2-3 years	no removal	no removal; clogging problems may occur
Products yielded	low value (hyacinths)	moderate value (duckweed)	moderate value (reed)	moderate value, wide variety (reed or wood)
Harvesting	difficult	easy	difficult	easy
Public health risks	moderate	moderate	moderate	low
Environmental value	low mosquito problems, hyacinth is weed in the tropics	low odor problems may occur	high	moderate

3 Evaluation of Project Feasibility: Methods and Criteria

The evaluation of the alternatives will consist of a financial cost-benefit-analysis and a checklist for the evaluation of impacts which are hard to express in financial terms. These impacts include effectivity; technical and regulatory feasibility; social impacts; and environmental impacts. The judgment of the feasibility of a reuse project should be based on a comparison with other possible solutions for the problems stated, using a set of criteria which regards all results caused by the implementation of the project. Treatment costs have to be balanced against public health risk and agricultural risks.

The feasibility of a reuse project depends on two values: the "sanitary" value and the "agricultural" value. The "sanitary" is the value of diminishing (treatment to prevent) contamination of natural resources, and can be estimated by the value of alternative treatment systems. The "agricultural" is the value of providing an alternative water resource for agriculture and can be estimated by the price of existing resources.

In the past many projects have emphasized financial criteria, while smaller projects have been aiming at environmental or social objectives, without taking into consideration the financial factors. Both kind of projects have not met expectations. A remarkable consensus has been emerging in recent years for managing water resources on an efficient, equitable and sustainable basis. At the 1992 Dublin International Conference on Water and Sustainable Development the idea was established that fresh water is a finite and vulnerable resource and should be recognized as an economic good (DICWSD, 1992; Serageldin, 1994).

3.1 Cost-Benefit-Analysis: Financial Aspects

A cost-benefit-analysis lists all costs and benefits per alternative, and thus provides a price label for each alternative. The economic criterion is a maximum benefit-cost-ratio. A wastewater reuse project is not likely to make a profit. It could, however, generate less costs than the sum of a separate treatment system and a separate irrigation scheme.

Costs and benefits are stated in US dollars, because this currency is internationally accepted. The life span of all alternatives is 20 years, after which the investment do not have a rest value. The interest rate is set at 24%; inflation at 21%. The real interest rate is 3%. The inflation rate in Cali over the past five years was 21%, and it is expected that this inflation will stay the same in the coming years.

The net present value method is used in order to take the time value of money into account. In this method the value of all expenditures and revenues in the past and in the future are determined at one definite point and time. For a series of annual payments in the future the net present value is given in equation 4 (Brouwer, 1993; Blokland and Trifunovic, 1994). The net present value at the first of May 1996 will be calculated for all costs.

$$W_h = A_{pay} * \frac{1-(1+r)^{n}}{r} \tag{4}$$

where: W_h = net present value [US$];
A_{pay} = annual payment [US$];
r = real interest rate [-];

The benefit-cost-ratio is the quotient of the total benefits and the total costs (see equation 5). The cost-benefit-ratio is 1, if total benefits equal total costs. If the benefit-cost-ratio is smaller than 1, the costs are larger than the benefits and the investment is loss-making. If the benefit-cost-ratio is more than 1, the benefits outweigh the costs and the investment is profitable (Brouwer, 1993).

$$B/C\text{-}ratio = \frac{\Sigma\ benefits}{\Sigma\ costs} \qquad \textbf{(5)}$$

The cost-benefit-analysis will only regard financial (investment and annual) costs. Investment costs consist of the acquisition of land, and the contraction costs for the construction of the system. Annual costs include energy costs, operation, and maintenance costs. Opportunity costs (the cost of water as an input) are not rated in the analysis. In irrigation the opportunity costs are a considerable fraction of total costs, especially in situations of water scarcity.

3.2 Qualitative Checklist

I have chosen to use a checklist in which an aspect can be rated desirable (+), no influence (0) and unwanted (-). A checklist consists of a list of impacts on one axis and quantitative and qualitative conclusions on the other axis. The qualitative checklist is introduced to account for impacts of the reuse scheme, which cannot be expressed financially.

3.3 Criteria for Checklist

Five categories of criteria are distinguished: effectivity, technical feasibility, regulatory feasibility, social preferability, and environmental preferability. These categories are discussed in the following paragraphs. The categories are based on the classification for the judgment of feasibility into four categories of Huckin (1991): effectivity, technical feasibility, affordability and preferability. Affordability is not discussed, as it is already discussed in the cost-benefit-analysis.

3.3.1 Effectivity of Proposed Solution

The term effectivity refers to the extend to which the implementation of a project solves the problems it is supposed to solve. The objective of agricultural wastewater reuse is two-sided. Reuse diminishes the contamination of natural water sources by treating wastewater and provides an alternative water source, and so diminishes depletion of scarce natural water resources.

Effectivity has to be judged on both objectives:

- Does contamination of natural water sources diminish after implementation of the project?
- Does depletion of natural water sources diminish after implementation of the project?

3.3.2 Technical Feasibility

Technical feasibility refers to the availability of resources that are necessary for the implementation of the project. These resources include technology, (skilled) labor, materials, and natural resources.

In order to judge technical feasibility the following questions have to be answered:

- Does the treatment concept have a solid theoretical basis which guarantees a secure construction and operation?
- Can the wastewater considerations be integrated into a distribution system in a secure way?
- Are skilled labor and manpower available for the implementation of the project?
- Are all materials for the construction of the system present or obtainable in Cali region?

3.3.3 Regulatory Feasibility: Institutional and Legal Aspects

Regulatory feasibility refers to the presence of institutions and regulations that influence the implementation of the project.To judge regulatory feasibility the following questions must be answered:

- Which laws and regulations exist, that the project has to obey? Do any of them forbid or complicate the implementation of the project?
- In which legislative units is the project located and does the project cross legislative borders?
- Does an institutional framework exist, which can implement the project? Or how can the institutional framework be established?

3.3.4 Preferability: Social Aspects

Preferability refers to the fact whether and the reasons why a solution is better than or preferred over other possible solutions. The social aspects of the reuse scheme include the risks for public health involved in the alternatives, the acceptance of reuse in the Valle de Cauca, benefits and nuisance for local communities and the effects of centralization of the irrigation system for farmers.

In order to judge social preferability the following questions have to be answered. Will the implementation of the alternative:

- create a public health risk?
- be socially accepted in the Valle de Cauca?
- create jobs and/or income for local communities?
- create infrastructure of services which can be used by local communities?
- offer education to local community members?
- occupy or affect areas which are of major significance for human settlement, agriculture, animal husbandry or similar?
- be self-sufficient; How is looked at ways to meet each need at a more local level?
- create nuisance conditions to people living nearby (smells, noise etc.)?
- be vulnerable for theft, vandalism, and terrorism?
- be socio-economically attractive through the yield of profitable products?
- be manageable for the farmers?

3.3.5 Preferability: Environmental Impacts

Environmental preferability refers to the fact whether the environmental impacts of a solution are better than or preferred over those of other possible solutions. The various aspects to judge environmental impacts are obtained from an overview of the Directorate General of International Cooperation of the Dutch Ministry of Foreign Affairs, which is used to list impacts of projects which they finance in other countries (DGIS, 1990).

The following aspects will be judged. Will the project:

- lead to the development of an unwanted disposal site, which displaces original wildlife?
- occupy or affect areas which are of major significance for human settlement, agriculture, animal husbandry or similar?
- occupy or affect areas which support animal or plant life worthy of protection or especially vulnerable ecosystems?
- affect areas with historic remains or landscape elements of importance to the population?
- lead to pollution of air, water or soil?
- drain rivers or change the flow of water in such a way that it creates considerable changes for the environment and the utilization of natural resources?
- require intensive use of non-renewable energy sources?
- lead to felling of trees for fuel etc. which is larger than the rate of growth?
- create major demands on other forms of infrastructure?
- cause a noticeable reduction in the flow of nutrient elements or fish production?
- lead to substantial waterlogging or salinity of cultivated or cultivable land?
- create a risk of spread of disease?

4 Pre-Irrigation Treatment for Cañaveralejo Plant Effluent

This chapter discusses the pre-irrigation treatment of the primary effluent of the Cañaveralejo plant. The pre-irrigation treatment makes the primary effluent of the Cañaveralejo plant suitable for its application: irrigation.

In the first paragraph (§4.1) the requirements of the pre-irrigation system are determined by comparing the present water quality with the water quality desired for irrigation. A pre-irrigation treatment system, which meets these requirements, is selected in §4.2. The location of the treatment is determined in §4.3. The selected system is designed in §4.4. A no pre-irrigation treatment alternative is discussed in short in §4.5. The final two paragraphs list the costs (§4.5) and the benefits (§4.6) of the treatment system.

4.1 Requirements of Pre-Irrigation Treatment System

The desired removal in the pre-irrigation system compares the desired water quality with the present water quality before pre-irrigation treatment (see equation 6). The latter is the primary effluent of the Cañaveralejo plant. Characteristics of the effluent of the Cañaveralejo plant are estimated using data of similar plants, performance expectations of primary treatment and literature values. The desired water quality results from general guidelines and local circumstances.

$$E_R = \frac{C_0 - C_e}{C_e} * 100\ \% \qquad \textbf{(6)}$$

where: E_R = removal efficiency [%];
C_0 = (present) influent concentration [mg/l];
C_e = (desired) effluent concentration [mg/l];

4.1.1 Desired Effluent

Important parameters for wastewater reuse for irrigation are the discharge, and the concentrations of BOD, TSS, nutrients, pathogens, salinity, specific ions, and trace elements. The desired water quantity and quality result from the application of general guidelines for irrigation water to local circumstances.

In the past the approach to wastewater has been to set universal standards and then to raise the funds to finance the required investments. This approach is turning out to be financially infeasible even in rich countries. A new approach has to been found in which a tradeoff is made between costs and water quality (Serageldin, 1994).

The characteristics of the Valle de Cauca are important factors in the assessment of the treatment required for wastewater reuse. Colombia is a developing country which cannot afford large expenditure for complete wastewater treatment. However, public health and long-term soil conditions should be protected. The sugarcane industry in the Valle de Cauca is well established, and will only accept a water that does not harm their crops. On the other hand farmers irrigate at present with water from the river Cauca, which at times is as dirty as primary effluent.

Discharge

The design flow of the pre-irrigation treatment in order to meet water demand is 475 l/s (41,000 m^3/day). The influent flow is 500 l/s, the effluent flow is 450 l/s. Ten percent of the influent flow is lost due to seepage.

A 10% seepage loss is accounted for in the design. Seepage in a lined pond system is low. Seepage only occurs at inlet and outlet structures, and is not a function of the surface area of the wetland. Water gains and losses due to evaporation and rainfall are negligible (see §4.4.7).

A central storing facility, integrated in the treatment system, is designed to obtain an intermittent flow during irrigation operation hours. The outflow of the treatment and storage system has to be an intermittent flow during 12 hours per day from Monday to Saturday of 900 l/s.

The effluent of the macrophyte-based treatment system will serve to irrigate the 936 ha irrigable area between the Rio Cauca and the Rio Fraile, which is defined in §5.2.1. The peak water requirement of the sugarcane is 384 l/s (see §5.3). To meet this water requirement in 6 days a week, a minimum flow of 450 l/s is required. On Sundays water is neither used nor stored.

Gross pollutants: BOD and TSS
A conservative limit of 50 mg BOD/l is set. Setting a standard for BOD-concentrations is difficult as several sources contradict each other on this subject: some do not set any limits, whereas others suggest maximums as low as 20 mg/l (US Department of the Interior, 1966; Culp and Hinrichs, 1981; Arthur, 1983; Shelef et al., 1987).

In this furrow irrigation system the suspended solids maximum of 50 mg TSS/l is set in order to avoid sedimentation of canals and ditches. Suspended solids can reduce already low soil infiltration rates, cause clogging of equipment and sedimentation problems in canals. Surface water often contains far higher concentrations of sediments (up to 30 g/l) than domestic wastewater (200-500 mg/l) (National Technical Advisory Committee, 1972).

Nutrients
The design of the pre-irrigation system aims at a nitrogen concentration between 10 and 20 mg/l. The maximum phosphorus concentration is 4 mg/l, this guideline is adopted from Ayers and Westcot (1985). This is a tradeoff between agronomic benefits and extra maintenance costs.

Nutrients in irrigation water can cause growth of weeds in canals which results in extra maintenance costs for the distribution system. Nutrients also generate agronomic benefits by fertilizing land and thus increasing yields. Fertilizers can be saved when applying nutrient rich irrigation water. Very high nutrient concentrations cause crop damage, and pollute groundwater.

Nitrogen concentration should be between 10 and 20 mg/l. Sugarcane is tolerant for nitrogen concentrations up to 30 mg/l. Irrigation practices can avoid damage in the ripening growing stage by not irrigating after a certain date. Nitrogen levels should be kept low in order to lower maintenance costs. Ayers and Westcot (1985) suggest values of 5 mg/l in critical circumstances, while the National Technical Advisory Committee (1972) states a maximal nitrogen concentration of 45 mg/l.

Pathogens
No faecal coliform limit is set, based on the WHO guidelines. Maximum helminth eggs concentration is 1 egg per liter.

The microbiological guidelines of the WHO of 1989 (table 45, appendix VII) classify sugarcane in category B, which are restricted crops which are not eaten raw (cereal crops, industrial crops, fodder crops, pasture and trees). The WHO does not specify a faecal coliforms limit for these crops, but states a maximum helminth egg concentration of 1 egg per liter.

Salinity
Maximum electric conductivity of the effluent is 1.7 dS/m. This guidelines is derived from the FAO guidelines, presented by Ayers and Westcot (1985). The salinity guidelines from the FAO are accepted as the most comprehensive and are used throughout the world (see figure 5, §2.3.2). Investigation of CENICAÑA of electric conductivity for varieties used in the Valle de Cauca supports the FAO value.

The sodium adsorption ratio of water is related to the electric conductivity, and a SAR limit should be set according to the relation discussed in §2.3.2 (see figure 5).

Specific Ions

A maximum chloride concentration of 107 mg/l is adopted from the FAO guidelines, as chloride affects soils and plants and is present in wastewater.

Ayers and Westcot (1985) provide FAO guidelines for four specific ions: sodium, bicarbonate, boron and chloride (see table 46, appendix VII). Sodium and bicarbonate are no major parameters when assessing irrigation water quality, and boron is only found in very low concentrations in wastewater. Chloride greatly affects soils and plants, and is present in wastewater.

Trace Elements

The design includes the following trace elements limits: 2.0 mg/l of zinc, 0.2 mg/l of nickel, 0.01 mg/l of cadmium, 5.0 mg/l of lead, and 0.20 mg/l of copper. These values are used in large parts of the world.

Most countries (i.e. USA, Colombia, and Saudi-Arabia) use a set of guidelines similar to the FAO guidelines, presented in Ayers and Westcot, 1985 (see table 47, appendix VII). The set of guidelines consists of a long list of trace elements, of which only zinc, nickel, cadmium, lead and copper are present in wastewater in harmful concentrations. It should be noted that these threshold limits are intended to protect even the most sensitive plants from harmful effects: if these guidelines are exceeded for any element, a second evaluation based on crop and local circumstances has to be made (Page and Chang, 1985a).

4.1.2 Influent

It is important to investigate the quality of the wastewater of the Cañaveralejo plant as it is a mixed wastewater. The wastewater of the Cañaveralejo plant is a weak wastewater of mainly domestic origin. The weakness of the wastewater is caused by the high water use per person in Cali. A small industrial area discharges on the sewer system of the Cañaveralejo plant. The Cañaveralejo plant is described in appendix VI.

The wastewater treated in the Cañaveralejo plant will be a combination of four sources: Cañaveralejo pumping station, Navarro pumping station, Aguablanca pumping station (which is not yet constructed), and the Colector General (see figure 10). The fact that those flows are not yet connected and samples are only taken in one of the four sources complicates estimating the quality of the wastewater.

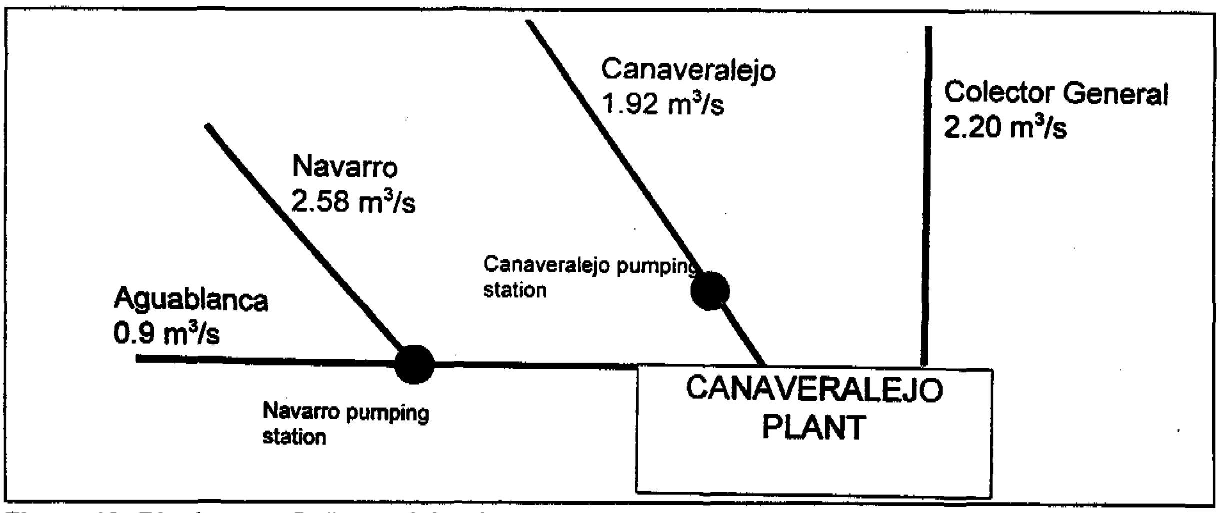


Figure 10: Discharges Cañaveralejo plant

The second problem when estimating wastewater quality of the Cañaveralejo plant is that the wastewater is not purely domestic. A small industrial area, which is located near the center of Cali, discharges its wastewater on the Colector General. Surveys of the kind of industries discharging on the Cali sewage systems do not exist. Although the national and provincial government have a large set of rules for industrial discharges, those rules are not yet enforced (CVC, 1976; Ministerio de Salud, 1984).

The following sources are available to calculate or estimate the quantity and quality of the effluent of the Cañaveralejo plant (table 3):

1. Design parameters of the Cañaveralejo plant: including discharge, BOD, TSS, nitrogen and phosphorus concentrations;
2. Measurements of raw wastewater of the Cañaveralejo plant: including BOD, TSS, nitrogen, phosphorus, electric conductivity, chloride, cadmium, chromium, copper, iron, manganese, nickel, lead, and zinc;
3. Effluent data from other wastewater plants, notably Vivero plant in Cali, and data from Medellin and Bogota: including BOD, TSS, nitrogen, and phosphorus concentrations;
4. Literature data on raw domestic wastewater quality: including BOD, TSS, nitrogen, phosphorus, faecal coliforms, helminth eggs, electric conductivity, boron, sodium, chloride, bicarbonate, aluminum, fluoride, manganese;
5. Literature estimates of removal efficiencies of preliminary and primary treatment.

Table 3: Availability of primary effluent data

	design (1)	measurements (2,3)		literature (4,5)		method	reliability
		Cañaveralejo	Other plants	quality of raw wastewater	removal		
Discharge	+					A	high
Gross Pollutants	+	+	+	+	+	A/B/C/D	high
Nutrients	+	+	+	+	+	A/B/C/D	high
Pathogens				+	+	D	medium
Salinity		+		+	+	B/D	medium
Specific ions		+			+	D	low
Trace elements		+		+	+	B/D	low

These data will have to be combined in order to assess effluent quality. There are several ways to combine these data (table 3):

A The effluent parameters of the design of the Cañaveralejo plant (Nitogoi, 1990) can be used without any further adjustment;

B The measurements of the raw wastewater of the future influent of the Cañaveralejo plant can be used in combination with removal data for preliminary and primary treatment given in literature;

C Measurements of other raw wastewaters at other plants can be combined with literature removal data;

D Literature data on the quality of raw wastewater can be combined with literature removal data;

With these methods the composition of the primary effluent flow of the Cañaveralejo plant, which is the influent flow for the pre-irrigation treatment, is characterized. The following parameters, which are of importance in agricultural reuse, are discussed: discharge, BOD, TSS, nitrogen, phosphorus, faecal coliforms, helminth eggs, electric conductivity, sodium adsorption ratio, chloride, zinc, nickel, copper, cadmium and lead.

Discharge
The maximum discharge available for reuse is the initial discharge after the first construction stage, which will be 7.6 m^3/s ($6.6 * 10^5$ m^3/d). EMCALI officially still plans on completion of the first stage of construction in 1998, but that this date will be reached, is very improbable, as construction has yet to begin and the design of the plant states a construction period of at least 4 years (Nitogoi, 1990).

Gross Pollutants: BOD and TSS
The BOD concentration is 106 mg/l and TSS concentration is 129 mg/l, these are the effluent concentrations after primary treatment of the design of the Cañaveralejo plant (Nitogoi, 1990). These parameters are also measured in the raw wastewater, the influent of the Cañaveralejo plant. The design values of raw wastewater (167 mg BOD/l and 224 mg TSS/l) are slightly lower than the measured values (194 mg BOD/l and 250 mg TSS/l).

Nutrients
Total kjeldahl nitrogen (TKN) concentration is 26 mg/l and total phosphorus (TP) concentration is 3 mg/l. These values derive from the design of the Cañaveralejo plant. When comparing the measured data with the design parameters of raw wastewater, nitrogen concentrations are very similar, but the phosphorus design concentration (3 mg/l) appears to be quite optimistic: it is consistently lower than average measured values (4.8 mg/l). As this is not an enquiry into the primary treatment provided by the Cañaveralejo plant, the design parameters of the effluent of the Cañaveralejo plant are used.

Pathogens
Primary effluent faecal coliform concentration is estimated at $5*10^7$ faecal coliforms per 100 ml, helminth egg concentration is estimated at 300 eggs per liter. These concentrations are based on literature, because pathogen data of the Cañaveralejo effluent or other raw wastewater sources in Colombia are not available.

Pathogen concentrations can be estimated well using literature values. Various resources (Arthur, 1983; Metcalf and Eddy, 1991; Mara et al., 1992; Mara, 1995) suggest a design value of $1*10^8$ faecal coliforms per 100 ml in raw wastewater. The number of helminth eggs in raw wastewater is normally in the range of 100 to 1000 per liter, 600 per liter is a good design estimate (Pescod and Mara, 1985; Mara et al., 1992). Primary treatment removes 25- 75% of faecal coliforms (Dean and Lund, 1981; Crook, 1985; Van der Graaf, 1995a), and removes 50-90% of helminth eggs (Dean and Lund, 1981; Crook, 1985). A presumed coliform removal of 50% leaves $5*10^7$ faecal coliforms per 100 ml; a helminth eggs removal of 50% leaves 300 helminth eggs per liter in the primary effluent of the Cañaveralejo plant.

Salinity
Electric conductivity of the primary effluent is estimated 0.70 dS/m. The electric conductivity derives from measurements and literature. The sodium adsorption ratio is in the range of 0 to 5.

EMCALI has not measured electric conductivity in the wastewater in Cali. In connection with this study a short series of measurements was taken in the Colector General, showing an average electric conductivity of 0.51 dS/m. A two-month measurements series of the raw wastewater at the Ginebra lagoon (see appendix VI) shows electric conductivity between 0.40 and 0.57 dS/m, average being 0.47 dS/m. Estimates for salinity in literature vary between 0.23 dS/m and 1.33 dS/m (D'Itri, 1981; Asano et al., 1985; Metcalf and Eddy, 1991). By combining literature and measured values raw wastewater electric conductivity of raw wastewater is estimated as 0.70 dS/m. Primary treatment does not remove salts, therefore the electric conductivity of primary effluent equals the value for raw wastewater.

The sodium adsorption ratio is assumed to be in the range of 0 to 5. This figure is based on the electric conductivity of the wastewater and the assumption that it has a normal relative sodium content. The sodium adsorption ratio is not measured in the wastewater if the Cañaveralejo plant. Very limited information is available in literature.

Specific ions
Chloride concentration is approximately 55 mg/l, based on literature and measurements. Eight measurements of chloride in the raw wastewater in the Colector General show an average concentration of 68 mg/l. Chloride in raw domestic wastewater is approximately 50 mg/l (Asano et al., 1985). An overall chloride concentration of 55 mg/l can be expected for the raw wastewater. The same concentration is present in the primary effluent of the Cañaveralejo, as primary treatment does not remove chloride.

Trace Elements
Trace elements concentrations are estimated based on measurements and literature: 0.092 mg/l of zinc, 0.016 mg/l of cadmium, 0.028 mg/l of lead, 0.115 mg/l of nickel, and 0.013 mg/l copper. These values are a combination of literature values and measurements.

Long series of measurements of zinc concentrations (43) and copper concentrations (31) in the Colector General are available. The analyses made of cadmium (10 measurements), nickel (10 measurements), and lead (11 measurements) are less representative. Metcalf and Eddy (1981) give some typical values for trace elements in domestic wastewater: the five trace elements occur in domestic wastewater in negligible concentrations. In this study trace elements removal in primary treatment is estimated to be zero, although it is believed that most trace elements are removed largely by sedimentation of solids in primary treatment (Page and Chang, 1985a).

Table 4: Performance requirements for macrophyte-based treatment system

			irrigation water guidelines	raw wastewater	primary treatment		pre-irrigation treatment	
					efficiency	effluent quality	required efficiency	required removal
general	discharge	l/s			influent flow = 500 l/s		effluent flow > 450 l/s	
	pH	units	6.5 - 8.4	6.7	-	6.7	-	-
	temperature	°C	-	25	-	25	-	-
gross pollutants	BOD	mg/l	50	167	37%	106	53%	56
	TSS	mg/l	50	224	42%	129	61%	79
nutrients	total nitrogen	mg/l	10-20	26	0%	26	23%-62%	6
	total phosphorus	mg/l	4	3	0%	3	0%	-
pathogens	faecal coliforms	#/100 ml	-	$1*10^{8}$	50%	$5*10^{7}$	-	-
	helminth eggs	#/l	1	600	50%	300	99.7%	299
salinity	EC_w	dS/m	1.7	0.7	0%	0.7	0%	-
	SAR	-	0 - 5	0-5	0%	0-5	0%	-
specific ions	chloride	mg/l	107	55	0%	55	0%	-
trace elements	cadmium	mg/l	0.010	0.016	0%	0.016	38%	-
	copper	mg/l	0.200	0.013	0%	0.013	0%	-
	nickel	mg/l	0.200	0.115	0%	0.115	0%	-
	lead	mg/l	5.000	0.028	0%	0.028	0%	-
	zinc	mg/l	2.000	0.092	0%	0.092	0%	-

This raises the question whether the primary effluent should be treated at all before irrigation: the irrigation area is very scarcely inhabited and the workers getting in touch with the irrigation water could be protected by design and operational measures. A no treatment alternative is discussed in §4.5.

4.1.3 Removal Requirements

Table 4, in which the primary effluent quality of the Cañaveralejo plant and the desired irrigation water quality are compared, shows that BOD, TSS, nitrogen, helminth eggs, and cadmium have to be removed.

4.2 Selection of Pre-Irrigation Treatment Concept

In order to identify the feasible pre-irrigation treatment concepts the treatment requirements for a particular project should be compared to the performance expectations of the various treatment concepts. All concepts that can satisfy these requirements should be included in further evaluation based on other properties. These properties include technical feasibility, area of land, yielded products, temperature, public health risks, integration of storage reservoirs, and construction and operation costs. Performance and characteristics of the various macrophyte-based treatment concepts are discussed in §2.4.7. An outline of the advantages and disadvantages of the macrophyte-based systems is presented in table 6.

Table 5: comparison of macrophyte-based systems

Parameter	hyacinth ponds	duckweed ponds	free water surface wetland	subsurface flow wetland
performance	+	+	+	+
experience	0	0	-	-
surface area	0	0	0	+
yielded products	-	+	+	+
performance under temperature	+	+	+	+
public health risks	0	0	0	+
integration of reservoirs	+	+	-	-
construction and operation costs	0	0	0	-

4.2.1 Comparison

Performance

The first selection of suitable pre-irrigation concepts has to be based on performance data: a concept that can not reach the required performance is not suited to treat the wastewater. The required performance as described in table 4 is compared to the typical performance of the various treatment concepts as presented in §2.4.7

All macrophyte-based treatment concepts do meet removal performances set for the pre-irrigation system. Nutrient removal is the most insecure part of all concepts, but the fairly low minimum removal performance can be reached in well-designed and well-managed systems.

Experience

Experience with all macrophyte-based treatment concepts is limited. Hyacinth and duckweed pond systems are upgraded waste stabilization ponds, which have shown to function properly under comparable circumstances. Wetland system performance is more insecure.

Waste stabilization pond systems are used throughout the tropics; an upgraded system with macrophytes on the final pond will not create a direct public health risk. The waste stabilization ponds in Ginebra in the Valle de Cauca function very well. Nutrient uptake by floating macrophytes is insecure, but low nutrient removal does not create a direct public health risk.

There are hardly any large scale wetlands in tropical conditions, and no wetlands operate in the Valle de Cauca. With the present knowledge and experience applying a subsurface flow wetland system creates a risk to public health, because clogging could occur, resulting in a surface flow. A subsurface flow wetland system, in which surface flow occurs, is an uncontrollable system with low removal parameters. Free water surface wetlands cannot clog, and design is more secure.

Area of Land
Subsurface wetlands occupy smaller areas than the other treatment systems for the same BOD removal. The surface area occupied by the treatment system should be as small as possible in order to limit investments. Land prices in the Valle de Cauca are fairly high, and close to the break-even point for natural treatment systems. Smaller treatment systems are influenced less by evaporation and rainfall.

Yielded Products
All concepts create a profitable yield. Hyacinth is a low value crop. Duckweed can be used as fodder for pigs, chicken, and fish. In the Valle de Cauca duckweed is used for pig fodder. Both subsurface flow and free water surface wetlands yield products which can be used in the production of crafts. Free water surface wetlands offer an additional environmental value. Harvest is easiest in duckweed ponds and subsurface flow wetlands.

Temperature
The temperature in the Valle de Cauca varies between 16 and 32 °C, all systems function very well at this temperature.

Public Health Risks
The public health risks of a well-functioning subsurface flow wetland are lower than those of other systems, as the wastewater flow is underground. A clogged subsurface flow wetland with a superficial flow, on the other hand, is more dangerous than other systems.

Integration of Storage Reservoirs
In a macrophyte-based pond system the final pond can function as a reservoir as well by varying the water level. This decreases the area of land needed for the combined treatment and storage system. The excavated volume is lower in combined treatment and storage pond systems (see figure 11).

Construction and Operation Costs
The construction of wetlands and separate reservoirs involves more excavation than pond systems (see figure 11). The high price of gravel in the Valle de Cauca increases the investments for subsurface flow wetlands. Operation costs are similar for all treatment concepts.

4.2.2 Conclusion

The duckweed pond treatment concept is selected as a pre-irrigation treatment. An alternative subsurface flow wetland is designed in appendix VIII.

Duckweed ponds are a technical feasible system, which combines good performance, low construction costs, low public health risks, and simple operation. Hyacinth ponds are similar to duckweed pond, but the products yielded are less interesting and more difficult to harvest. Free water surface wetlands combine higher public health risks with a low technical feasibility. Subsurface flow wetlands offer the optimal combination of costs, risks and operation, but experience is very limited.

An alternative design for a subsurface flow wetland is made in appendix VIII. Experience on subsurface flow wetlands is too limited to put them into practice at present, but the idea is potential for the future. In this case the subsurface flow wetland turns out considerable more expensive, because of the need of gravel and excavation.

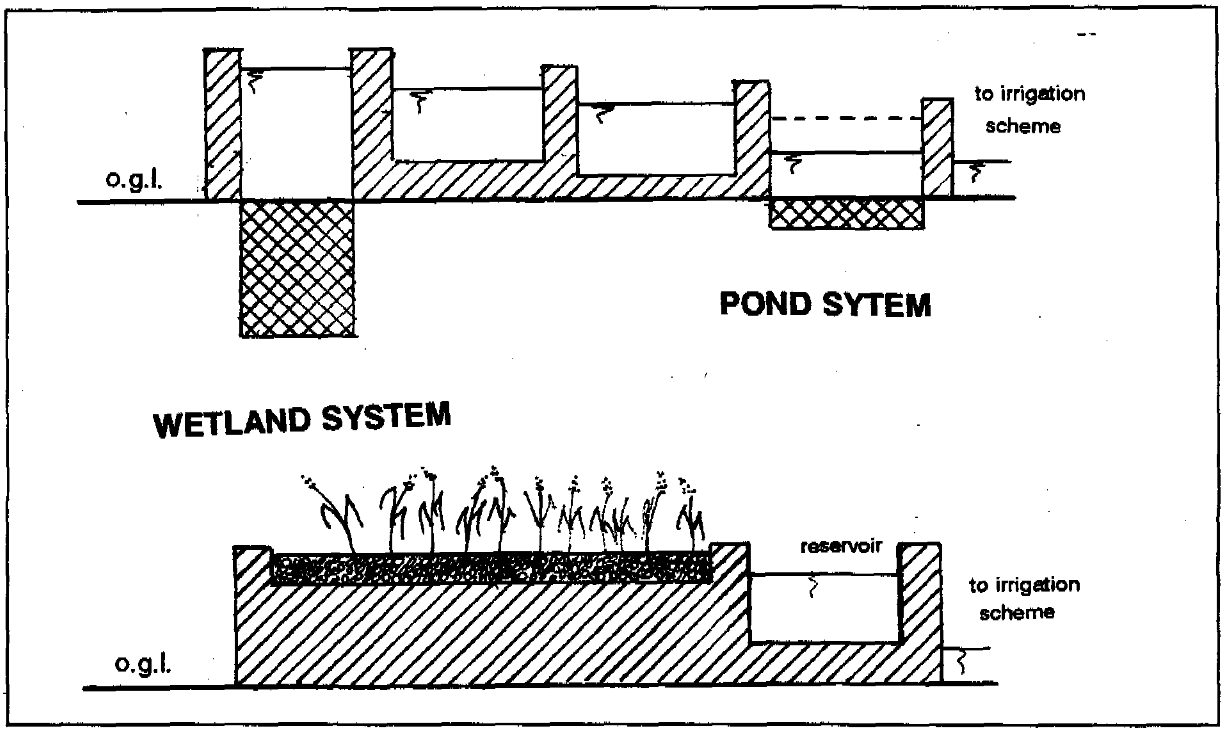


Figure 11: excavation of preapplication treatment system, including reservoirs

4.2.3 Design parameters

The design temperature is 25 °C, which equals the design temperature of the Cañaveralejo plant. The design discharge is 475 l/s. The other design parameters (presented in table 6) are the pollutants which have to be removed (required removal > 0 in table 4).

Table 6: design parameters for pre-irrigation system

parameter	symbol	unit	influent	effluent	removal
Discharge	Q	m^3/day	-	41,000 (475 l/s)	-
Temperature	T	°C	25	25	-
BOD	C	mg/l	106	≤50	53%
TSS	C	mg/l	129	≤50	61%
Nitrogen	C	mg/l	26	≤20	23%
Helminth eggs	C	#/l	300	≤1	99.7%
Cadmium	C	mg/l	0.016	0.01	38%

4.3 Location of Pre-Irrigation Treatment System

The site of the pre-irrigation treatment system should be located near the railway bridge over the River Cauca, which is nearest to the outlet of the Cañaveralejo plant. This reduces transport costs and hydrological head. The elevation of the Eastern border of the River Cauca is 4 meters lower than the outlet of the Cañaveralejo plant, so the wastewater can be transported across the river by gravity.

The pre-irrigation treatment will be constructed between the Rio Cauca and a little stream called Zanja Curiche, North of the railway line. The treatment system should be located behind the dike of the river Cauca to prevent flooding. The water is used at the Northern side of the railway line after treatment, and the location at the Northern side of the railway reduces construction of canals. Climate, soil characteristics and land prices are about constant in the area. Topography of the land behind the dike is uniform, which is favorable for the construction of wetlands and pond systems, as it reduces earthwork.

The area at the Eastern bank of the River Cauca is in another municipality (Palmira) than the source of the wastewater (Cali). This may generate regulatory and institutional problems. The site does not interfere with wildlife. Trucks have access to the site by a dirt road in good condition on top of the dike of the River Cauca. Materials can also be transported to the site by railway as well.

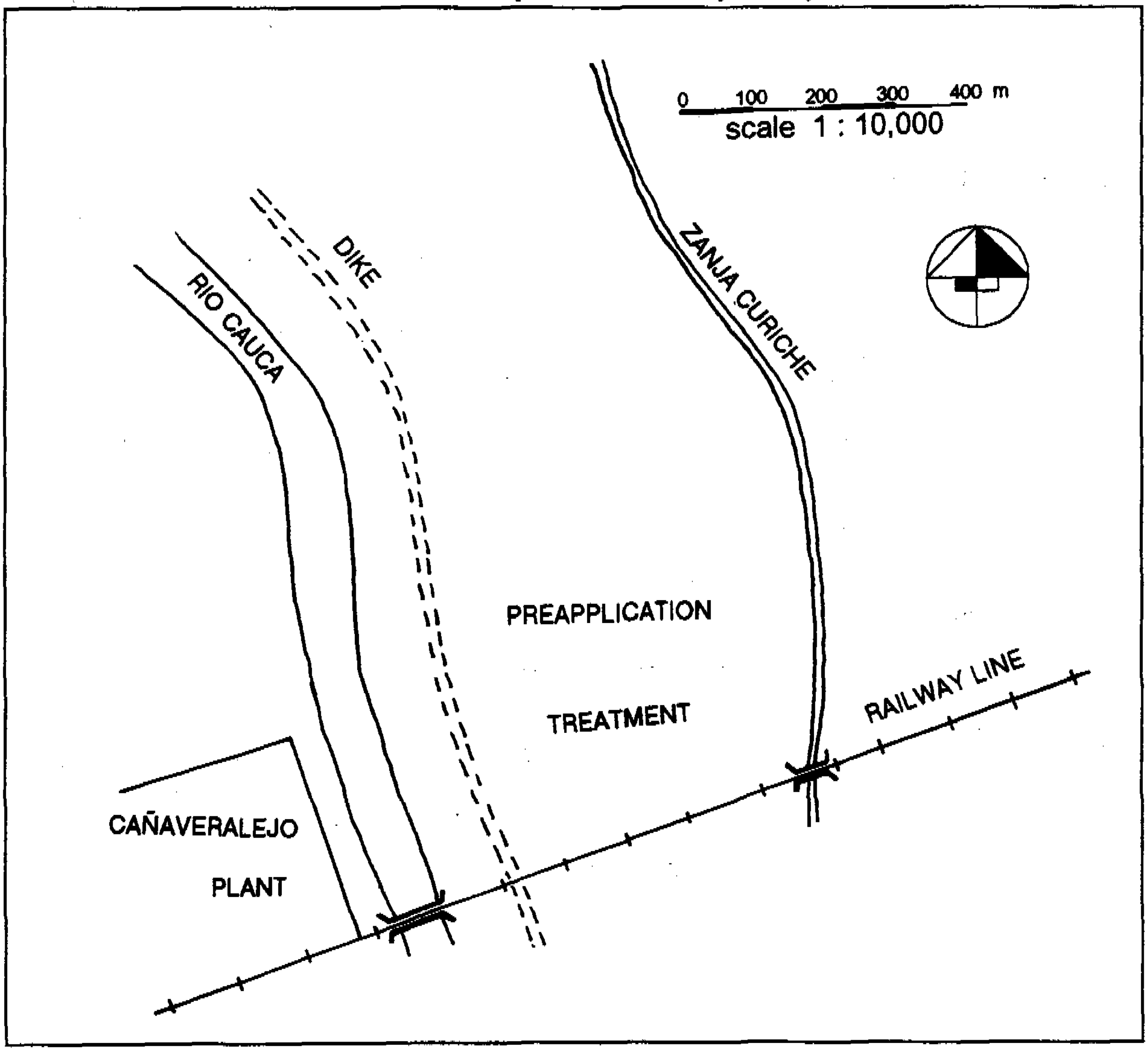


Figure 12: Location of pre-irrigation treatment

4.4 Duckweed Pond System

This paragraph describes the design of a duckweed pond system. First the pond system is characterized (§4.4.1) and the design concepts are determined (§4.4.2). The stages of the actual design are: the design of the anaerobic pond (§4.4.3), the facultative pond (§4.4.4), and the maturation ponds (§4.4.5); the integration of storage reservoirs (§4.4.6); the description of the influence of rainfall and evaporation (§4.4.7), the layout of ponds (§4.4.7), and the design of structures and piping (§4.4.9). The performance of the pond system and he suitability of the effluent for irrigation are discussed in §4.4.10. $4.4.11 describes the management, operation, and maintenance of the pond system.

4.4.1 Characterization of Pond System

Waste stabilization pond systems comprise a single series of anaerobic, facultative and several maturation ponds, or several such series in parallel. Anaerobic and facultative ponds are designed for BOD removal. Duckweed maturation ponds are designed for pathogen removal and nutrient removal.

The construction depth of the anaerobic pond is 4 m. The pond will fill up with sludge until a depth of 2.5 m, when it will be desludged. Anaerobic ponds normally are between 2 and 5 m deep and receive raw or settled wastewater. Anaerobic ponds function much like open septic tanks. They are loaded with high BOD loading rates of more than 300 g/m^3*d. The ponds are completely devoid of dissolved oxygen, and the suspended solids settle to the bottom of the pond, where they undergo anaerobic digestion.

The design depth of the facultative pond is 1.5 m. Facultative ponds are mostly between 1 and 2 m deep and receive either raw wastewater, settled wastewater, or effluent of anaerobic ponds. Facultative ponds have a lower anaerobic zone and an upper aerobic zone where oxygen for bacterial metabolism is largely provided by the photosynthetic activity of microalgae.

This pond system is designed with two subsequent maturation ponds of the same size in each series of ponds. The maturation ponds will be covered with duckweed. The design depth is of 1 m. Maturation ponds with floating macrophytes are typically 1 and 1.5 m deep. Maturation ponds principally reduce the number of excreted pathogens and nutrients. Maturation ponds show less vertical stratification than facultative ponds, and are well oxygenated throughout the day.

Lining

The permeability of the soil at the pond site requires the installation of an impermeable liner at the bottom of the ponds to prevent groundwater contamination and water losses. The basic infiltration rate of the Juanchito soil is 4.6 mm/hour, Mara et al. (1992) advise lining if the basic infiltration rate of the soil is more than 0.36 mm/hour.

Geotextile, a sheet material that is available in Columbia, is selected to line the ponds. Geotextile is strong, impermeable, and smooth to prevent attachment. Other possible materials are compacted in situ soils (permeability less than 10^{-6} cm/s), bentonite, asphalt, synthetic butyl rubber, or plastic membranes (Steiner, 1989).

4.4.2 Design Concepts

Design methods of anaerobic and facultative ponds are based on a loading rate approach. The loading methods account for the influence of temperature on system performance, by varying loading rates with temperature. Design concepts for duckweed maturation ponds are scarce. Most design manuals for waste stabilization pond systems are written by the same authors and are based on the same experiences (Arthur, 1983; Mara, 1989; Pescod and Mara, 1989; Mara et al., 1992; Reed et al., 1995).

A volumetric loading approach is used for the design of the anaerobic ponds. In anaerobic ponds BOD removal is achieved by sedimentation of settleable solids and anaerobic digestion in the resulting sludge layer, so the volume of the pond is more important than the surface area.

An areal loading approach is used for the design of the facultative ponds. In secondary facultative ponds, the remaining non-settleable BOD is oxidized by heterotrophic bacteria. The oxygen for this process is obtained from aeration through the surface of the pond. Surface area is therefore more important than volume.

The main function of maturation ponds with floating macrophytes is the removal of pathogens and nutrients. Faecal coliforms are removed in facultative and maturation ponds. Faecal coliform removal is modelled with first order plug flow kinetics. Helminth eggs are removed by sedimentation. Helminth eggs removal can be predicted by an empirical equation, based on experiences in Brazil, India and Kenya (Mara et al., 1992). Nitrogen uptake by duckweed and nitrogen removal in ponds without macrophytes can be estimated using empirical equations (Reed et al., 1995).

4.4.3 Anaerobic Pond

Sizing of anaerobic ponds is based on BOD removal. Overloading an anaerobic pond causes odor problems, underloading causes aerobic conditions and incomplete digestion of sedimented solids. A conservative empirical approach, based on the permissible volumetric BOD loading (λ_v), is used here (Mara, 1989).

The design loading is 300 g/m^3*d for temperatures over 20 °C. The removal of BOD at this loading rate is 60%. The volume of the anaerobic pond is 14,278 m^3 (equation 7). The surface area of the pond is 5715 m^2, and is determined by dividing the volume by the depth of the pond (see equation 8). The hydraulic retention time is the quotient of the volume and the discharge (see equation 9), and is 0.35 days (Mara et al., 1992).

$$V_{ap} = \frac{Q * C_0}{\lambda_v} \qquad \textbf{(7)}$$

where:
- V_{ap} = volume of anaerobic pond [m^3];
- Q = discharge [m^3/d];
- C_0 = influent concentration [mg/l];
- λ_v = volumetric loading rate [g/m^3*d];

$$A_{ap} = \frac{V_{ap}}{y} \qquad \textbf{(8)}$$

where:
- A_{ap} = surface area of anaerobic pond [m^3];
- y = depth [m];

$$t = \frac{V}{Q} \qquad \textbf{(9)}$$

where:
- t = hydraulic retention time [d];

Helminth eggs removal can be estimated per pond by an empirical relationship (equation 10), based on the hydraulic retention time (Mara, 1994). Pathogen concentrations are presented in table 7.

$$C_e = C_0 * 0.41 * \exp(-0.49 * t + 0.0085 * t^2) \qquad \textbf{(10)}$$

where:
- C_e = effluent concentration [mg/l];

Table 7: Concentrations in pond system

place	BOD	TSS	faecal coliforms	helminth eggs	nitrogen
	mg/l	mg/l	#/100 ml	#/l	mg/l
influent anaerobic pond	106	126	$5*10^7$	300	26
influent facultative pond	42	?	$5*10^7$	103	26
influent first duckweed pond	11	?	$4.1*10^6$	18	25.7
influent second duckweed pond	<11	?	$5.0*10^5$	4	19.4
final effluent	<11	<126	$6.1*10^4$	<1	15

4.4.4 Facultative Pond

Facultative ponds are also designed on BOD removal: this time an areal BOD loading (λ_s) is used. The surface area of the facultative pond is 48,984 m^2 (equation 11). The hydraulic retention time is 1.82 days (see equation 9).

$$A_{fp} = \frac{Q_o * C_{ap}}{\lambda_s} \tag{11}$$

where: A_{fp} = surface area of facultative pond [m^2];
C_{ap} = concentration in effluent anaerobic pond [mg/l];
λ_s = areal loading rate [g/m^2*d].

The faecal coliform concentration of the facultative pond effluent is $4.1*10^6$ per 100 ml, this is estimated using first order plugflow equation 12 (Reed et al., 1995).

$$C_e = C_0 * \frac{1}{[1 + t * (k_t)]} \tag{12}$$

$$k_t = 2.6 * (1.19)^{(T_w - 20)} \tag{13}$$

where: k_t = temperature dependent rate constant [d^{-1}];
T_w = mean water temperature [°C];

Nitrogen removal in facultative and maturation ponds without floating macrophyte is very low, and can be estimated using empirical relationship 14 (Mara et al., 1992). The effluent nitrogen concentration is 25.7 mg/l.

$$C_e = C_o * (1 + (5.035*10^{-3} * \frac{A}{Q})) * \exp(1.540 * (pH - 6.7)) \tag{14}$$

4.4.5 Maturation Ponds

Two subsequent maturation ponds of the same size are constructed. The size of the maturation ponds is based on both helminth eggs removal and nitrogen removal. The required helminth eggs removal is 94%, the required nitrogen removal is 22%.

The minimum hydraulic retention time for the required helminth eggs removal is 1.16 days. This implies a surface area of 45,000 m^2 each (see equation 10). The effluent faecal coliform concentration is $6.1*10^4$ per 100 ml (equation 12).

Reed et al. (1995) give an empirical relation between the hydraulic loading rate of a floating macrophyte pond and the nitrogen removal (see equation 15 and 16). The hydraulic loading rate of both duckweed maturation ponds is 9,000 m^3/ha*d. The effluent nitrogen concentration after two subsequent 45,000 m^2 duckweed ponds is 15 mg/l.

$$HLR = 10{,}000 * \frac{Q}{A} \tag{15}$$

$$C_e = C_o * [1 - (\frac{760}{HLR})^{\frac{1}{1.72}}] \tag{16}$$

where: HLR= hydraulic loading rate [m^3/ha*d];

4.4.6 Integration of Storage Reservoirs

A variation of the water level of the final duckweed pond of 0.45 m provides the set night storage capacity. The required storage volume is 20,000 m^3 to store a flow of 450 l/s for 12 hours.

4.4.7 Evaporation and Rainfall

Water gains and losses in the pond system derive from evapotranspiration, precipitation and seepage. Evaporation slows the water and increases hydraulic retention time, but on the other hand concentrates the pollutants in the wastewater. Rainfall has the opposite effect, it shortens hydraulic retention time and dilutes pollutant concentrations.

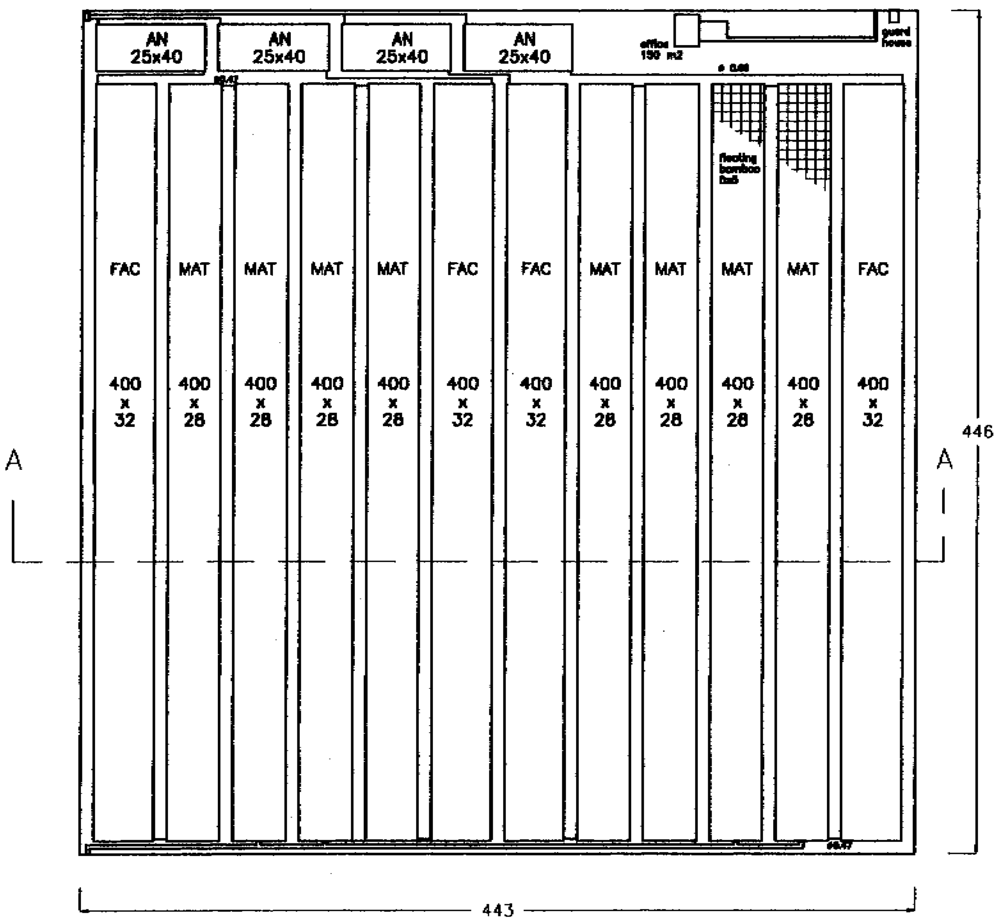
Water gains from evaporation and rainfall can be estimated in two ways: the average flow approach which presumes linear evaporation and rainfall (Reed et al., 1995), and the more complex actual retention time approximation (Kadlec 1989). The simpler average approach is used here, as only very small gains and losses of the influent flow occur due to evaporation and precipitation. Literature does not quantify the dilution of wastewater by rainfall or concentration by evapotranspiration.

The month in which evaporation is lowest and rainfall is highest, the wettest month, is the measure for water gains and losses. The wettest month in the area is April, evaporation (E_o) is 129.2 mm/month and average rainfall (P) is 139 mm/month (based on Rio Cauca station, appendix IV). The flow gain is 6193 m^3/month equals 206 m^3/day, which is 0.5% of the daily flow (equation 17). The water gain is so small that it is not taken into account for the design.

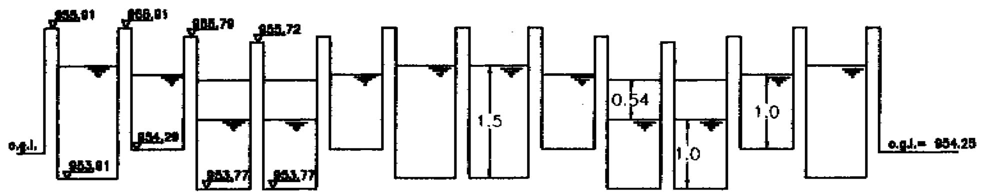
$$\Delta Q = A * (P - E_0) \tag{17}$$

$$Q_e = Q_0 + \Delta Q \tag{18}$$

where: ΔQ = flow gain [m^3/d];
P = average precipitation [m/month];
E_0 = evaporation [m/month];
Q_e = effluent discharge [m^3/d];
Q_0 = influent discharge [m^3/d];



Layout of Pond System



Cross-Section A – A

Legend:
AN = Anaerobic Pond
FAC = Facultative Pond
MAT = Maturation Pond

Interpond Connection

Figure 13
Waste Stabilization Pond System
Canaveralejo Reuse Scheme
Meike van Ginneken
Date: 8 – 1996   Scale 1:4,000

4.4.8 Layout of Ponds

The pond system serves a large discharge and four series of ponds in parallel are built in order to increase operational flexibility. The pond geometry minimizes hydraulic short-circuiting and excavation. Ponds are rectangular, length-to-width ratios vary. Inlet and outlets are located in diagonally opposite corners of the pond to avoid short-circuiting (see figure 13). The ponds fit in the 450 m wide strip of land between the River Cauca and the Zanja Curiche. The total surface area is 450 x 450 = 19.8 hectares

The anaerobic ponds are located at the Northern end of the other ponds and are 58 m x 25 m. Anaerobic ponds have length-to-width ratios of 2.3:1 to avoid sludge banks forming near the inlet.

The water flows in a zigzag through the long facultative and maturation ponds. Facultative ponds have a length-to-width ratio of 1:12.5 to approximate plugflow conditions (Mara et al, 1992; Mara, 1994). The facultative ponds are 400 m long and 32 m wide. Duckweed maturation ponds have a length-to-width ration of 1:14. They are 400 m long and 28 m wide. In order to avoid piling of duckweed by wind, floating baffles of split bamboo are laid in a grid of 5 m x 5 m.

The vertical position of the pond system is designed to minimize net excavation. This means that the volume excavated should equal the volume elevated above original ground level (see equation 19). Average original ground level is 954.25 m. Freeboard of all pods is 0.5 m. The maximum inflow level is 956.74 m (the head of the wastewater after crossing the River Cauca). The minimum outflow level is 954.56 m (the water level at the top of the irrigation canal system. The optimum situation is an outflow level of 954.57 m out of the ultimate duckweed pond, and an inlet level of 956.06 m. For this situation the excavation balance is presented in table 8, and the cross-section is presented in figure 13. For head losses in the pond system see table 9.

$$V_{embankment} = V_{ponds} \qquad (19)$$

Table 8: excavation pond system

pond	depth of pond	water level elevation	bottom pond* (1)	area ponds (2)	top dike* (3)	area dikes (4)	excavation **
	m	m	m	m^2	m	m^2	m^2
anaerobic	4	955,74	-2.51	5800	1.98	3070	-8449
facultative	1,5	955,51	-0.24	51200	1.75	14800	13760
maturation I	1	955,	0.16	44800	1.65	9120	22307
maturation II	1.45	954,89	-0.81	44800	1.13	5120	-28147
					TOTAL NET EXCAVATION		-529

* in relation to the original ground level
** excavation of pond is (1)*(2) +(3)*(4)

4.4.9 Structures and Piping

Piping within the pond system as well as the transport across the River Cauca can be calculated using the formula of Strickler (equation 20, rewritten in equation 21). The hydraulic radius of a pipeline is one fourth of the diameter. The head losses in the pipes consist of friction losses and local losses (see equation 22, 44). Maximum velocity in all pipelines is 2 m/s.

$$Q = 0.312 * k_s * D^{2\frac{2}{3}} * s^{\frac{1}{2}} \tag{20}$$

$$H_f = s * L = [\frac{Q}{0.312 * k_s * D^{8/3}}]^2 \tag{21}$$

where:
k_s = Strickler roughness coefficient [$m^{1/3}/s$];
D = diameter [m];
s = slope [-];
H_f = friction head loss [m];
L = length of pipe [m];

$$H = H_f + H_l = s * L + k * \frac{v^2}{2 * g} \tag{22}$$

where:
H = total head loss [m];
H_l = local head loss [m];
k = local head loss coefficient [-];
v = velocity [m/s];
g = 9.81 m^2/s;

Piping in Treatment System

In the pond system head losses consist of friction head losses and local head losses in pipes and the head loss over the ponds (H_p). Pipelines are made of concrete (Strickler roughness coefficient is 75 $m^{1/3}/s$). The slopes of the pipes are 1‰, and 2‰. Total head loss in the system is 1.39 m. The head losses and pipes of the pond system are presented in table 9.

Table 9: Piping in pond system

pipe	$Q_{required}$	L	i	D	k	Hf	Hl	Htotal
	l/s	m	1e-3	m	-	m	m	m
crossing	500	200	4	0.63	4	0.8	0,56	1,36
to anaerobic pond	250	266	1	0,68	0	0,266	0,00	0,32
anaerobic - facultative	250	150	1,0	0,53	1,2	0,15	0,03	0,25
facultative - maturation1	125	10	2,0	0,47	0,8	0,02	0,03	0,12
maturation1 - maturation2	125	10	2,0	0,47	0	0,02	0,00	0,52
outlet canal	900	380	0,2	open canal		0,19	0,00	0,19

Crossing the River Cauca
The Cañaveralejo plant is located on the West bank and the pre-irrigation treatment site is located on the East bank of the river Cauca, so the wastewater has to cross the river. This can be done with a steel siphon (k_s = 90 $m^{1/3}/s$), which can either be placed on the bottom of the river or be attached to the railway bridge. The latter solution is cheaper, and therefore preferred.

The discharge to be transported is a continuous flow of 500 l/s. Regardless of the water demand, the full discharge is treated, and surpluses are disposed of.

The width of the river is 100m, the forelands and the dike are another 80m wide. Including some extra length for curves, the total length of the siphon is 200m. The flow can be by gravity as there is 2.13 m head available, between the outlet of the Cañaveralejo plant (elevation 958.10 m; Nitogoi, 1990), and the inlet of the treatment system (elevation 955.97 m). Total local losses coefficient (k) is estimated to be 4.

The full discharge can be transported in one pipeline. The pipes available on the local market are 16, 18, 20, 24, and 28 inches in diameter. A 24 inch pipe with a slope of 4‰ is selected (see table 54).

Inlets and Outlets of Ponds
Inlet and outlet structures should be simple, and permit samples of the pond to be taken with ease. Inlets should discharge well below the liquid level so as to minimize short-circuiting. Outlets of all ponds should be protected against the discharge of scum by the provision of a scum guard. By installing a variable height scum guard, optimizing the take-off level once the pond is in operation, is permitted (Mara et al., 1982).

A neyrtec C2 distributor allows for large variations in upstream water level at the outlet of the final maturation ponds. This variation is caused by the fact that these ponds serve as reservoirs as well. The outlet structure can be a neyrtec distributor or a combination of an AVIO gate and a neyrtec distributor. The Neyrtec C2 series is the cheapest solution available. Each of the four neyrtec distributor should have a width of 0.23 m to allow for a flow of 225 l/s. The neyrtec C2 distributor has a headloss of 0.24 m.

4.4.10 Performance of pond System and Suitability of Effluent for Irrigation
Based on the calculations carried out above the effluent quantity and quality of the duckweed pond system is estimated and the suitability for irrigation is discussed.

Discharge
The effluent flow from the pond system meets the irrigation water demands. The short-term and long-term variations in the discharge of domestic wastewater does not influence the flow in the duckweed pond system, as only part of a larger wastewater flow is used. The effluent flow is constant during a year. This is an advantage for irrigation, that normally camps with scarce water resources in dry times, when peak irrigation water demands occur. The intermittent irrigation flow required for the irrigation project is obtained by storing the water in the final duckweed pond.

Gross Pollutants: BOD and TSS
An estimate 90% BOD removal of the pond system results in an effluent BOD concentration of 11 mg/l. This removal is mainly obtained in the anaerobic and the facultative ponds (Mara et al., 1992). This BOD concentration does not create problems or restrictions in the use of the wastewater, and the selection of the field application system.

The TSS concentration in the effluent is hard to estimate, as on the one hand solids present in raw wastewater are removed, but on the other hand new algae are introduced. The duckweed on the maturation ponds decreases the suspended solids concentration in the effluent. TSS concentrations in raw wastewater are already much lower than the level found in surface water used for irritation. The suspended solids concentration does not create problems in the irrigation scheme.

Table 10: water quality of effluent

			desired effluent	influent	duckweed pond system	
					effluent	removal
general	discharge	l/s	450	500	450	-
	pH	units	6.5-8.4	6,7	6,7	-
	temperature	°C	-	25	25	-
gross pollutants	BOD	mg/l	50	106	11	90%
	TSS	mg/l	50	129	no data	no data
nutrients	total nitrogen	mg N/l	20	26	15	42%
pathogens	faecal coliforms	#/100ml	-	3e+07	6e+04	99.8%
	helminth eggs	#/liter	1	300	1	99.7%
salinity	ECw	dS/m	1.7	0,7	0,7	0%
	SAR		0-5	0-5	0-5	0%
specific ions	Chloride	me/l	3.0	1.6	1.6	0%
trace elements	cadmium	mg Cd/l	0.010	0.016	0.003	80%
	copper	mg Cu/l	0.200	0.013	0.001	80%
	nickel	mg Ni/l	0.200	0.115	0.023	80%
	lead	mg Pb/l	5.000	0.028	0.000	80%
	zinc	mg Zn/l	2.000	0.092	0.012	80%

Nutrients

The nitrogen concentration in the effluent of the duckweed pond system is 15 mg/l. Furrow irrigation is used for field application: clogging of equipment will not occur, but extra consideration should be paid to the maintenance of canals.

High nitrogen levels are beneficial during early growth stages, but may cause yield losses during the later growing stages. In early growing stages the rates of nitrogen fertilizer supplied to the crop can be reduced by an amount equal to that available from the water supply (Ayers and Westcot, 1985). The irrigation practice in the Valle de Cauca plans the last water gift in the tenth growing month, therefore the risk of excess nitrogen through irrigation in the final growing stages is negligible.

Pathogens

Helminth eggs removal is nearly complete, and the concentration is lower than 1 egg per liter. This meets the WHO requirement for restricted irrigation.

The concentration of coliforms is $6*10^4$ per 100 ml. This concentration does not allow for irrigation of crops eaten raw: the application of the wastewater should be controlled. The coliform concentration does not create problems for the irrigation of sugarcane.

Salinity

Salinity and sodium adsorption ratio are not influenced by the treatment in the duckweed pond system. The salinity and sodicity of the effluent do not damage crops and soils.

Specific Ions
Chloride concentration of the pond effluent is 1.6 mg/l, the pond system does not remove chloride. The chloride does not pose problems for the use as irrigation water.

Trace Elements
The effluent cadmium concentration is 0.0026 mg/l, with an estimate trace element removal of 80%. Trace elements are removed in the anaerobic pond, and later by duckweed uptake in the maturation ponds. The principal heavy metal removal mechanism in anaerobic ponds is by precipitation. Heavy metals removal of anaerobic reactors is over 90%, measured values for anaerobic ponds vary between 50 and 95% (Mara and Wills, 1994). Duckweed maturation ponds also remove trace elements from wastewater. If the duckweed is to be reused as animal fodder, this is a disadvantage, because the value of the yielded product decreases.

The effluent trace elements concentrations are estimated very preliminary, but even if the presumptions made are only partly just, the pond effluent still meets the very stringent guidelines. The trace element concentrations in the pond effluent are 4 (cadmium) till 900 (lead) times lower than the general guidelines. The concentrations in the wastewater are estimated using rough literature values and short measurements series. The removal of trace elements in the primary treatment system (0%) and the pond system (80%) are estimated prudently. Further investigation into the amount of trace elements in the raw wastewater is necessary before putting into practice a reuse scheme.

4.4.11 Management, Operation, and Maintenance
The maintenance requirements of ponds are simple, but they must be carried out regularly to avoid odor, fly and mosquito nuisance. Larva-eating fish, such as *Gumbusia* or *Peocelia,* are introduced to avoid mosquitoes breeding. Routine maintenance tasks are (Reed et al., 1995):

- removal of screenings and grits from the inlet works, and accumulated solids in the inlets and outlets;
- cutting the grass on the embankments and repair of any damage to the embankments;
- removal of floating scum and harvesting of floating macrophytes from the surface;
- spraying the scum on anaerobic ponds;
- repair of any damage to external fences and gates;

Table 11 shows the staff for a waste stabilization pond system of this size (Mara et al., 1992). Once a waste stabilization pond system is established a routine monitoring and evaluation program should be introduced, so that its performance can be verified and the actual quality of its effluent established.

Table 11: staffing of pond system

job title	amount
foreman/supervisor	1
mechanical engineer	1
laboratory technician	2
assistant foreman	2
laborers	10
driver	2
watchman	10
TOTAL	28

Anaerobic ponds need to be desludged every two years, when they are filled with sludge for approximately one third (Mara et al., 1992). The power needed for desludging is $2.5*10^8$ J, which is 70 kwh (equation 23), given a pump efficiency of 0.7, a density of sludge of 1025 kg/m^3, a volume of

3500 m^3, and a head of 5 m (out of the pond, up a bit and friction losses.

$$P_{ower} = \frac{g * \rho_{sludge} * V * H}{\eta} \quad \textbf{(23)}$$

where: P_{ower} = power [J];
ρ_{sludge} = density of sludge [kg/m^3];
η = efficiency [-];

4.5 No Pre-Irrigation Treatment Alternative

The use of primary effluent of the Cañaveralejo plant for irrigation without further treatment poses some risks for public health, crops, and soils. On the other hand it saves large expenditure on wastewater treatment. The decision whether to provide pre-irrigation treatment is an economic decision. Risks have to be balanced against costs. The no pre-irrigation alternative is evaluated in the feasibility evaluation (chapter 6) as alternative 2.

Risks
The primary effluent of the Cañaveralejo plant does not meet the guidelines for BOD, TSS, nitrogen, helminth eggs, and cadmium (see §4.1.3). This poses problems to public health (helminth eggs) and crops and soils (cadmium).

The BOD concentration of the primary effluent (106 mg/l) does not pose serious problems. Standards for BOD in irrigation water contradict each other. The TSS concentration (129 mg/l) does increase maintenance costs slightly, but do not create risks to crops, soils or public health. The nitrogen concentration (26 mg/l) does increase maintenance costs, but decreases the use of fertilizers. The nitrogen concentration does not create a public health risk through the pollution of groundwater (see §5.3.1).

The helminth eggs concentration in the primary effluent (300 eggs/l) does create a public health risk to the workers in the sugarcane fields. Consumers of sugarcane are not affected, as this crop is processed and cooked before consumption. If farmers use the irrigation water for other crops, consumers are in danger. Protective measures can reduce the public health risk. These measures include protective clothing for workers, and information about the location of fields and risks to the local community.

The cadmium concentration (0.016 mg/l) is only slightly above the guideline set by the FAO (0.010 mg/l). The measurements of cadmium and other trace elements are, however, so rough, that problems with crops and soils could occur. The wastewater should before investigated more thoroughly before putting into practice a reuse scheme with primary effluent.

Storage
Primary effluent requires additional treatment prior to storage, to prevent reservoirs from becoming septic (Smith et al., 1985). Night reservoirs cannot be used. The discharge of the influent will be intermittent: during the night and the weekend no water will be transported across the river.

Crossing
For direct application of primary effluent irrigation water has to be transported across the river during daytime, when land is irrigated. The outlet of the Cañaveralejo plant is 958.10 m, the inlet of the irrigation system is 954.56 m. The head available for the crossing is 3.54 m. For flexibility a number of smaller pipes is preferred. and an intermittent flow of 900 l/s if the primary effluent is used directly for irrigation. For the intermittent flow of 900 l/s three pipes of 18' with a slope of 7.5‰ are the cheapest combination.

4.6 Cost Estimate

In order to judge the affordability of the various alternatives in a later stage, the costs of each alternative for treatment have to be determined. Costs consist of investments and operation and maintenance costs. The costs of a duckweed pond system, the costs of an activated sludge installation, and the costs of the crossing in the no treatment alternative are discussed here. Appendix IX lists prices in the Valle de Cauca, including the sources they are derived from.

Investments of Pond System

Investments for the construction of a pond system include the acquisition of land, excavation and liner, and the construction of structures and pipes, and the construction of facilities for the staff.

The largest investment for a pond system is the acquisition of land. The unit cost of land is hard to estimate, as it depends largely on demand. Agricultural land in the Valle de Cauca costs about US$2-5 per m^2; prices in Cali are above US$20 per m^2 (CVC, pers.comm., Ingenio Mayagüez, pers.comm.). The influence of the presumed land price is further discussed in chapter 6.

Table 12: investments of duckweed pond system

item	unit	unit cost	amount	price
Land acquisition	m2	$10	200,000	$2.000.000
Building of ponds				
excavation	m3	$3	110,000	$370,000
liner	m2	$1	175,000	$170,000
clearing and grubbing	ha	$493	20	$10,000
Structures/piping				
crossing (600m 24" steel piping + labor)				$20,000
concrete piping (26")	m	$97	800	$80,000
concrete piping (20")	m	$47	210	$10,000
concrete piping (18")	m	$35	77	$3,000
canal (900 l/s)	m	$36	450	$16,000
canal (250 l/s)	m	$18	1100	$20,000
interpond connections	#	$50	16	$800
neyrtec C1 module	m	$28,000	0.96	$27,000
labor	d	$5	500	$2,500
Facilities				
office building	#	$140,000	1	$140,000
guard house	#	$5,000	1	$5,000
fencing	m	$15	1800	$26,000
TOTAL				$2,900,000

The digging of the ponds involves over 100,000 m^3 of groundwork, and 175,000 m^2 of lining with geotextile. Prices for excavation and geotextile are known (see appendix IX). A large volume has to be excavated, although net excavation is very small.

The investment for structures and pipes are US$150,000. This includes the steel piping to cross the river Cauca, 1100 m of concrete piping, and 500 m of open canal, 16 interpond connections, 4 neyrtec C1 modules. These amounts are based on table 9 and figure 13.

The costs of the office, the guard house, and the fencing are US$165,000. The office facilitates for pond staff and irrigation scheme staff (see §5.2.4).

Annual Costs of Pond System

The annual costs of the pond system are US$200,000. Annual costs of the pond system include personnel, materials, costs of desludging, and maintenance of the crossing.

The personnel of the pond system is discussed in §4.4.11. Material costs are presumed to be 1% of the construction costs, as there are hardly any mechanical parts in the system. Desludging is discussed in §4.4.11. The maintenance of the crossing is 1% of the construction costs of the crossing.

Table 13: Annual costs of duckweed pond system

item	unit	unit cost	amount	price
Personnel				
foreman/supervisor	year	$11,048	1	$11,000
mechanical engineer	year	$8,406	1	$8,400
laboratory technician	year	$4,803	2	$9,600
assistant foreman	year	$7,085	10	$70,900
laborers	year	$2,557	2	$5,100
drivers	year	$2,401	5	$12,000
watchmen	year	$2,401	10	$24,000
materials	-	$29,000		$29,000
Desludging				
desludger hire	days	$2,501	1	$2,500
pumping of desludging	kwh	$0.071	35	$3
sludge disposal	m3	$14.65	1750	$25,600
maintenance of crossing	year	$197	1	$200
TOTAL				$200,000

Costs of Crossing for Reuse with Primary Effluent
The investment costs for 3 pipes with a diameter of 18' is US$35,000, The maintenance is estimated to be 1% of the investment, or US$350 per year. There are no treatment costs, when directly using primary effluent.

Cost of Activated Sludge Plant
The investment cost of activated sludge treatment of 500 l/s are estimated at US$6,000,000. Total annual costs are estimated at US$200,000 (see table 14). These estimates are based on the official EMCALI estimate for the Cañaveralejo plant.

The cost of activated sludge treatment is hard to estimate, as stated prices vary largely. The investments of the activated sludge stage (3.8 m^3/s) of the Cañaveralejo plant were estimated in 1992 to be US$42.5 million (EMCALI, unpublished). This figure has not been adjusted for inflation over the past years, and still is the official EMCALI estimate. In Europe the investments for an activated sludge treatment of 500 l/s (200,000 i.e., cost per i.e. is US$500) are approximately US$100 million. The annual operational costs are approximately US$1 million (Van der Graaf, 1995a.). The construction of a pilot activated sludge plant of 22 l/s in 1986 in Medellin costed US$100 million, the present operation costs are US$500,000 per l/s per year (department of public works, Medellin, pers.comm.)

Table 14: Annual costs of activated sludge treatment

item	unit	unit cost	amount	price
operation costs	m^3	$0.01	16,000,000	$150,000
sludge management	m^3	$15	3416	$50,000
TOTAL				**$200,000**

4.7 Benefit Estimate

The pre-irrigation system has the direct benefit of a profitable duckweed yield. Indirect benefits refers to the decrease in contamination by the implementation of the system. This can be estimated by looking at the price of alternative treatment options, and is discussed in chapter

Duckweed is used as a substitute for soya cake as a protein-rich pig fodder in small community projects in the Valle de Cauca. Based on experiences of CIPAV (Investigation Center for Sustainable Agricultural Production Systems) the following calculation of the price of duckweed can be made. Duckweed contains 5% of dry matter of which 35% is protein. Comparable protein sources for pig fodder (soya cake) cost Col$845 per kilo of protein, the price of the (protein in the) duckweed is 0.05*0.35*845 = Col$10.6 per kilogram (US$11 per ton). Data on the yield of duckweed per hectare vary: Reddy and DeBusk (1985) state a yield between 6 and 25 tons/ha per year, Janney et al. (1992) observed 13 to 38 tons/ha per year, and local experiments by CIPAV reached between 256 and 304 tons/ha per year. Estimating a yield of 50 tons/ha per year, a benefit of US$550 per year is calculated.

5 Predesign Irrigation System for Cañaveralejo project

This chapter presents the design of an irrigation system. The effluent from the pre-irrigation treatment in chapter 4 serves as the water resource. All stages of the design feature on water quality considerations. The irrigation area is described in §5.1. §5.2 characterizes the irrigation scheme. Water requirements are discussed in §5.3. The layout and the design of canals and structures follow in §5.4 and §5.5. An example of a tertiary unit design is developed in §5.6. The final part of the chapter lists the benefits (§5.7) and the costs of the irrigation scheme (§5.8).

5.1 Characterization of the Irrigation Area

The irrigation area is characterized by climate, soils and topography, the crop, the management and organization of farmers, the population, present irrigation practices and water resources.

5.1.1 Climate

The dominant climate is an Aaf-climate in terms of the Köppen's classification: A tropical forest climate with all months average above 18°C, warmest month above 22°C and rainfall in every month of the year. Climate contrasts within short distances are widely developed in the northern portion of tropical America, and the deep valley of the river Cauca has a complex climatic pattern, with various micro-climates (Rumney, 1967).

Average annual rainfall in the study area is in the range of 850 mm to 1100 mm, dry months being January/February and July/August, wet months being April/May/June and October/November. Evaporation is 1400 mm to 1600 mm annually. Humidity is high, between 70 and 80%. Average temperature is 24°C. Wind velocity does not exceed 2 m/s. A more detailed description of the climate is given in appendix IV.

The precipitation data of the Planta Rio Cauca station are used in this design. This series is the longest and most representative available in the area. Only open-pan evaporation is measured in the Valle de Cauca. Evaporation data of the Granja ICA are used, because this is the longest series available (see table 15, and figure 14).

Table 15: Evaporation and Rainfall

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Ann
Evaporation	144	136	150	130	124	120	139	148	144	139	123	132	1627
Rainfall													
average	51	74	115	139	115	74	41	52	66	127	104	77	1033
80% dep	18	22	63	84	58	32	13	12	26	83	62	39	512
effective	15	17	50	67	46	25	10	10	21	66	50	31	409
Deficit	-94	-62	-35	9	-9	-46	-98	-96	-78	-12	-19	-55	-594

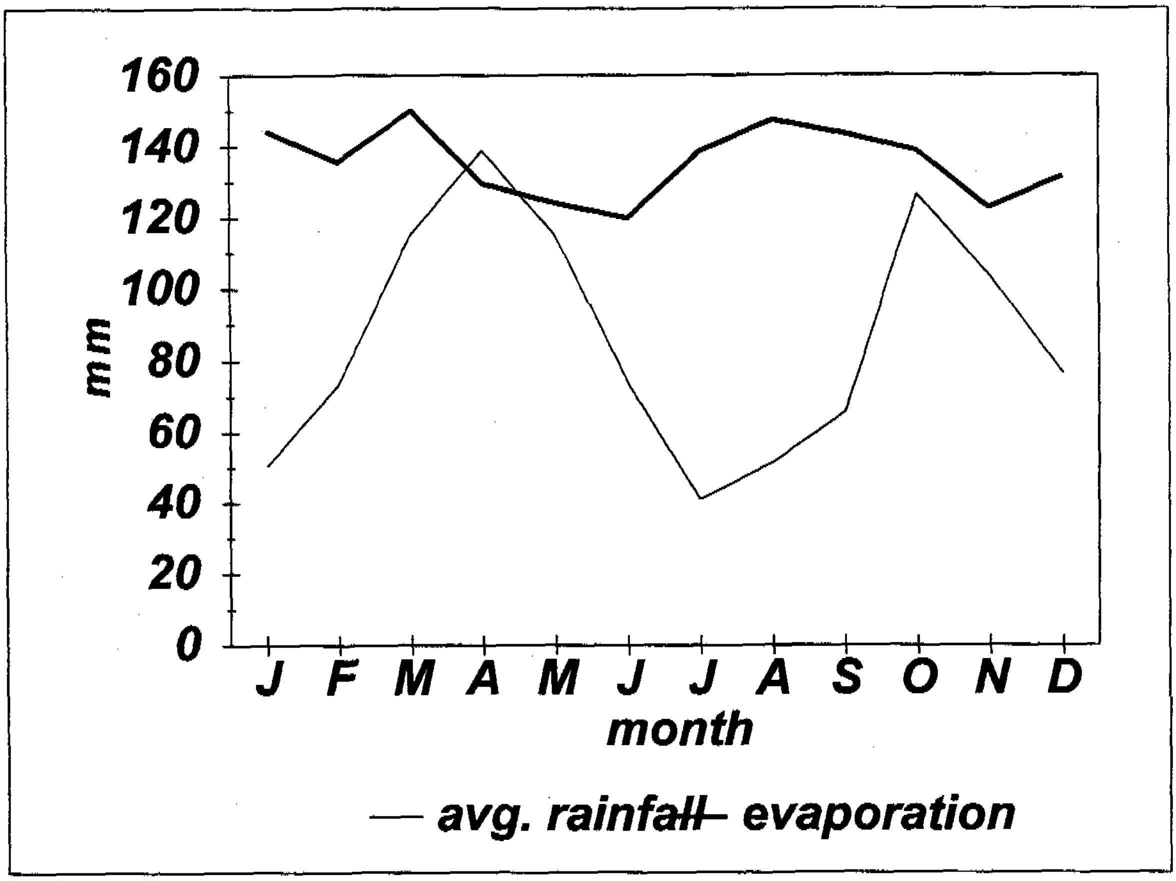


Figure 14: Evaporation and rainfall

5.1.2 Soils and Topography

In the Valle de Cauca alluvial deposits are scattered in narrow, irregular strips bordering the river Cauca and other streams. Four soil types (Juanchito soils, Marruecos soils, Gudualito soils, and Corintias soils) dominate the Juanchito-Guanabanal-Palmaseca area. They can be classified in three orders, according to the USDA soil taxonomy: mollisols, vertisols, entisols (Foth, 1978; CVC, 1980). In appendix V the soil types are described.

Information on the topography of the study area is incomplete and outdated. Topography and elevation is estimated based on the following data on the Juanchito-Guanabanal-Palmaseca:

- topographical maps, 1:25.000, from 1976;
- incomplete elevations maps, 1:10.000, contour line interval 1 m, from 1962;
- elevation maps of the farms Los Mangos and La Chachiporra, 1:2,500, contour line interval 0.25 m, not dated;
- soil maps 1:50.000, from 1985;
- land use maps 1:50.000, from 1992;

5.1.3 Crop: Sugarcane

Sugarcane (*Saccharum officinarum*) is the world's most important sugar crop. It is grown primarily for sugar (sucrose) but molasses, ethyl alcohol, and fiber (bagasse) are important by-products. Sugarcane is a perennial crop, with several ratoon (stubble regrowth) crops following the plant crop. 185.000 ha of sugarcane is grown in the Valle de Cauca. Yields are among the highest in the world. Growing stages of the crop can be rotated, because of the steady climate. The optimum harvest age in the Valle de Cauca is 13 months. Table 16 presents fertilizer needs of sugarcane in the Valle de Cauca (Jones et al., 1990; Torres, 1995).

Table 16: fertilizer needs of sugarcane

element	products	fertilizer need	
		first crop	ratoon crop
Nitrogen	urea / ammonia sulphate	100 kg N/ha	150 kg N/ha
Phosphorus	SFT / DAP	25-50 kg P_2O_5/ha	-
Potassium	KCl	25-50 kg K_2O/ha	-

The growth stages of a commercial sugarcane crop are unrelated to flowering, as it does not often occur in commercial fields. Jones et al. (1990) distinguish four growth stages: (1) germination and emergence, (2) tillering and canopy establishment, (3) grand growth, and (4) ripening. Doorenbos and Kassam (1986) divide sugarcane vegetative growth into seven stages each with their own crop coefficients for relating evapotranspiration to reference evapotranspiration values (see table 17). Studies of the Colombian national sugarcane research institute (CENICAÑA) set apart three growth stages: the tillering and canopy establishment stage (2nd tot the 4th month), the grand growth stage (5th till 10th month), and ripening (Torres, 1995). CENICAÑA crop coefficients are a combination of the relation between open-pan evaporation and reference evapotranspiration and the FAO crop factor (see table 17).

Table 17: Sugarcane crop coefficients

Development stages	length [d]	period [d]	k_c FAO[1]	k_c CENICAÑA
planting to 0.25 full canopy	30 - 60	1-30	0.40 - 0.60	-
0.25 - 0.50 full canopy	30 -40	31-60	0.75 - 0.85	-
0.50 - 0.75 full canopy	15 - 25	61-75	0.90 - 1.00	0.3
0.75 to full canopy	45 - 55	76-120	1.00 - 1.20	0.3
peak use	180 - 330	121-300	1.05 - 1.30	0.7
early senescence	30 - 150	301-330	0.80 - 1.05	-
ripening	30 -60	331-360	0.60 - 0.75	-

1. k_c values depend on minimum relative humidity and wind velocity

Sugarcane has a deep rootzone and yield does not drop dramatically as a result of occasional water stress. Therefore long irrigation intervals are permitted (Plusquellec et al., 1994). Some varieties tolerate moderate salinity and seasonal flooding, but good drainage and salinity management are required for high yields (Ayers and Westcot, 1985; Jones et al., 1990). Sugarcane is moderately sensitive to salinity. Decrease in crop yield due to increasing salinity is: 0% at EC_e 1.7 mmhos/cm. 10% at 3.3, 25% at 6.0, 50% at 10.4 and 100% at EC_e 18.6 mmhos/cm (see figure 4, §2.3.2). Gomez and Torres (1995) have investigated salt tolerances in the Valle de Cauca, and found similar values for various varieties.

5.1.4 Management and Organization of Farmers

Farmers are organized in *ingenios*. An *ingenio* is a sugarcane factory, which also cultivates sugarcane on farms owned by the *ingenio* or rented from individual farmers. Some farmers cultivate their own crop, and have a long-term contract to sell their cane to an *ingenio*. The 13 Ingenios of the Valle de Cauca are organized in ASOCAÑA, which manages CENICAÑA, a research institute for sugarcane.

5.1.5 Population

The total population of the area between the Rio Cauca and the Rio Fraile is approximately 3,000 people: 1,000 in Caucaseca, 1,300 in Las Dolores and 700 in farms and houses scattered throughout the area. Caucaseca is a settlement South of the Cali-Palmira road, adjacent to the Rio Fraile. Caucaseca had 225 inhabitants in 1988 (Bocanegra and Giraldo, 1989); on a field visit the village appeared to have grown to approximately 1,000 inhabitants. Las Dolores is a mainly industrial area on the border of the River Cauca, just North of the Cali-Palmira road. It had 1307 inhabitants in 1988, which still seems a realistic figure (Bocanegra and Giraldo, 1989).

5.1.6 Present Irrigation Practices in the Valle de Cauca

Irrigation in the Valle de Cauca is mainly furrow irrigation. High pumping costs forced the sugarcane producers to improve furrow irrigation practices and to look for other irrigation techniques, such as sprinkler irrigation and trickle irrigation. Although irrigation programming is improved, some farmers still do not take into consideration the relation soil-water-crop (Aristazabal and Zambrano, 1993).

Field Application Systems

Furrow irrigation systems have been improved in various ways in the past few years. Reduced furrow length has increased the uniformity of application and the field application efficiency. Alternating furrows (giving water only to one furrow out of two or three) has raised efficiency. The new alternating furrow technique does not only save spending on water supply, but also decreases labor costs. It is implemented in 50-60% of the sugarcane fields in the Valle de Cauca. Unimproved furrow irrigation systems in the Valle de Cauca have a field water application efficiency of 20 to 30%, alternating furrows systems have field application efficiencies of 40-50% (Torres, CENICAÑA, pers.comm.).

Sprinkler systems are used in 10- 15% of the Valle de Cauca, mainly at locations where furrow irrigation is not practicable. CENICAÑA found that up till now it is not economically feasible for most *ingenios* to switch to sprinkler systems (Cruz, CENICAÑA, pers.comm.).

Trickle irrigation is practiced in only 1-2% of the Valle de Cauca. CENICAÑA investigators found no difference in the production of sugar between fields irrigated with trickle irrigation or with alternating furrow irrigation. Water use is about 50% lower with trickle irrigation. With the present cost of water trickle irrigation is only feasible in areas where water is very scarce (Cruz, CENICAÑA, pers.comm.).

Conveyance and Distribution System

Farms in the Valle de Cauca have their own water supply, either from adjacent rivers or from wells. Therefore they are not used to sharing water. In the Valle de Cauca the quaternary units are in general between 4 and 11 ha: furrows are 100 to 120 m long, furrows are 1.5 m apart and 250 to 500 furrows are served by one quaternary canal (Caraval and Marmolejo, 1989).

Sharp-edged adjustable gates and broad-crested measuring weirs are the only structures in operation in the Valle de Cauca (Torres and Cruz, 1993). The absence of more sophisticated structures is due the minimal need for discharge measuring and regulating in the decentralized irrigation practices. Districto RUT in the Northern part of the Valle de Cauca is one of the few centralized irrigation schemes. In this scheme there is some experience with automatic downstream regulators but parts were stolen, and lack of experience reduced the operation possibilities. The regulators are now replaced for more robust structures. Theft and vandalism is a problem with all advanced equipment used in sugarcane irrigation in the Valle de Cauca.

Losses in tertiary canals are very high. CENICAÑA has developed a system to decrease losses drastically: the use of flexitubes and polyethylene pipes instead of open canals. This measure increases distribution efficiencies to 85%- 90% (Cruz et al., 1995).

A study on conveyance systems in an area, which is adjacent to the Juanchito-Guanabanal-Palmaseca area, showed efficiencies in primary and secondary canals of 51% (Caraval and Marmolejo, 1989). Causes for this low performance include long canal lengths, poor maintenance, occasional overflow due to excess discharges, bad construction of canals without taking into account soil characteristics, and deterioration of gates. Benitez (1994) quotes an average efficiency of primary canals in another part of the Valle de Cauca of 79%, and states that lining of canals is economically feasible because of high costs of pumping groundwater.

Primary canals are between 500 and 4000 meter long, discharges vary between 87 and 371 l/s. The length of tertiary canals is 500 to 700 meter, discharges are between 17 and 58 l/s (Caraval and Marmolejo, 1989).

Efficiencies

The combination of the low efficiencies, which are mentioned above, result in an overall efficiency of unimproved systems as low as 11% (see equation 24).

$$e_p = e_a * e_d * e_c = 30\% * 60\% * 60\% = 11\% \qquad \textbf{(24)}$$

where:
- e_p = overall efficiency [-];
- e_a = field water application efficiency [-];
- e_d = distribution efficiency [-];
- e_c = conveyance efficiency [-];

The *ingenios* have implemented water-saving measures in the past few years. The saving in water use is shown in figure 15. Overall efficiency for improved systems, using alternating furrow irrigation, lined primary and secondary canals and flexitubes in tertiary canals, is approximately 38%:

$$e_p = e_a * e_d * e_c = 50\% * 85\% * 90\% = 38\% \qquad \textbf{(25)}$$

On the long term CENICAÑA predicts that improved alternating furrow irrigation systems can reach a 60% efficiency:

$$e_p = e_a * e_d * e_c = 70\% * 90\% * 95\% = 60\% \qquad \textbf{(26)}$$

Storage

The great variation in rainfall from year to year is one of the problems faced in irrigation planning and design in the Valle de Cauca. A huge storage capacity is installed to prevent crop damage in times of drought and flooding in extremely wet times. Most *ingenios* count 60 to 70 reservoirs, volumes varying between 10,000 and 1,000,000 m^3. Reservoir efficiency is 98% per day. Depth of reservoirs vary between 1.5 and 1.8 m. Reservoirs are generally unlined, but small particles of suspended solids penetrate the walls and bottom and thus decrease permeability and seepage (Cruz, CENICAÑA, pers.comm).

Drainage

Drainage by gravity is possible most of the year. During periods of high water tables in rivers, the area can only be drained by pumping (Caraval and Marmolejo, 1989). In April and May 90% of the zone is affected by high water tables, the rest of the year 50% remains affected.

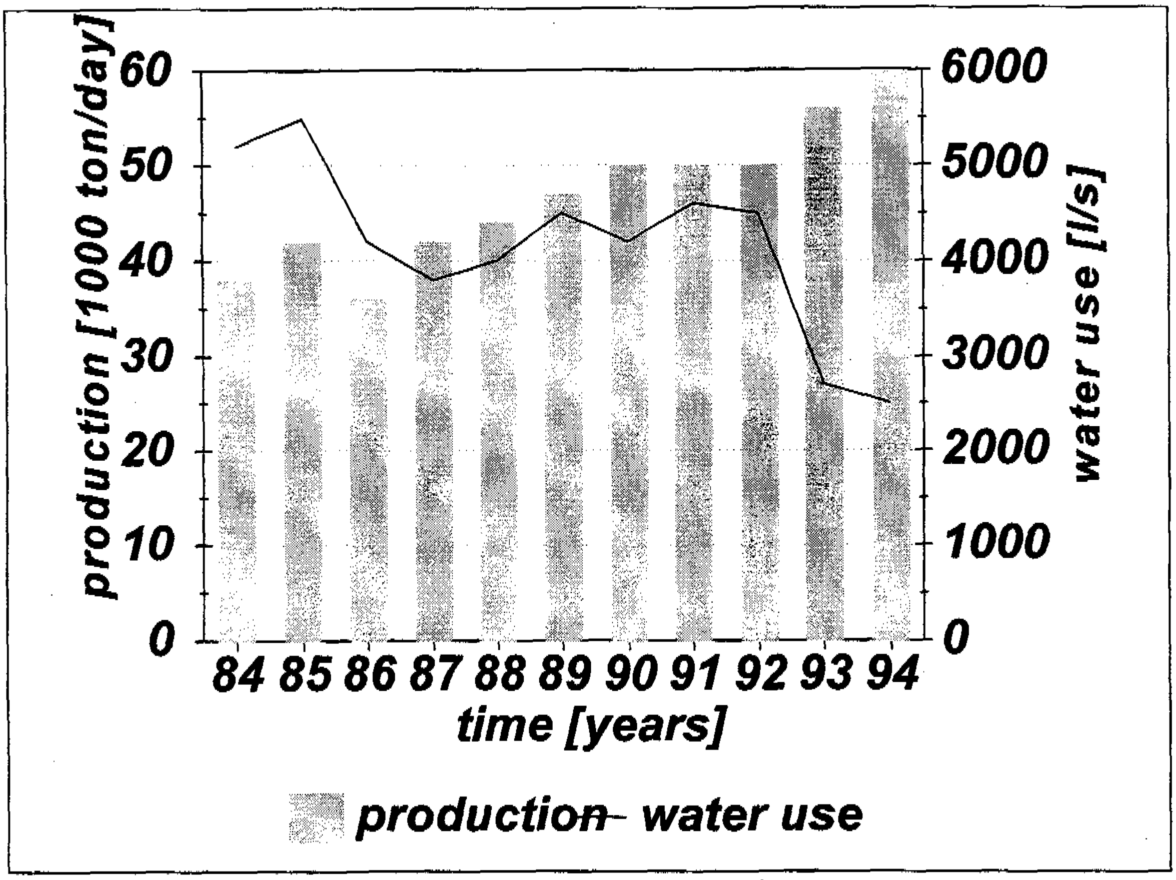


Figure 15: water use and the production of sugarcane 1984-1994

5.1.7 Water Resources

Present water sources are groundwater (pumped from deep wells) and surface water (obtained from rivers and streams). The ground water quality is suitable for irrigation. The surface water can cause clogging of equipment and sedimentation of canals.

The Valle de Cauca groundwater is not contaminated with trace elements and excess salt (Martinez, 1989). The depth of wells is related to their age: before 1965 wells of less 120 m were built, from 1965 on wells with a depth of 150-180 m were built and in the last few years several wells of more than 200m are constructed, reaching into aquifer B (see appendix III).

The water quality of the River Cauca is hardly suitable for irrigation. The River Cauca is a muddy stream, which has an average discharge of 200 m^3/s around Cali. At present nearly all domestic and industrial wastewater of the cities along the river are discharged untreated on the river. The CVC, the regional water resources council, measures pollutants in the Rio Cauca about 6 times per year at various sites. PH varies between 6.1 and 7.4. Faecal coliform levels in the river Cauca show wide variations, average being about $1*10^6$ per 100 ml, but sometimes rise to $1*10^{10}$ per 100 ml. Salinity of the river water is not a problem for irrigation, the highest measured electric conductivity is 0.36 dS/m. The river Cauca does not meet irrigation water quality criteria for some trace elements. Total solid concentrations in the river are high (up to 500 mg/l) (Martinez, 1989; CONE, 1995; CVC, pers.comm.).

The Rio Fraile is much smaller and cleaner than the River Cauca. The base discharge is 592 l/s. The Rio Fraile is not affected by salinity: average salinity at the bridge of the Cali-Palmira road is 0.19 dS/m. Other quality problems are not likely to occur as no wastewater is discharged upstream. The use of the Rio Fraile is divided as follows (Bocanegra and Giraldo, 1989):

- 0.5 l/s/ha for rice;
- 0,32 l/s/ha for sugarcane;
- 0.08 l/s/ha for fruit trees, corn and soya;
- 260 l/person/day for domestic use;
- 50 l/animal/day for livestock.

5.2 Characterization of the Irrigation Scheme

The principles on which the system is based have to be determined before planning the layout of an irrigation system. This irrigation scheme is productive, or crop-based irrigation: the objective of the irrigation management is to maximize the return per unit area. The irrigation scheme has to function by gravity.

Other principles include: the definition of the irrigable area, the field application system, the choice between a combined or separated system, the management, the water delivery schedule, the operation, and the control system. Based on these characteristics the efficiency of the system can be estimated. This paragraph discusses all those aspects.

5.2.1 Definition of Irrigable Area in the Juanchito-Guanabanal-Palmaseca zone

The irrigable area is 936 ha and is the part of the study area that can be used for irrigated agriculture. The irrigable area is determined by defining the geographical area, the useful agricultural area and the command area (see figure 16). The availability of data, topography, elevation and land use form the base for the definition of the irrigable area (Bergmann and Boussard, 1976).

The geographical area is the area of all farms and communes whose land is located within the irrigation perimeter, including agricultural area used and areas that are not under cultivation or not cultivable. The geographical area equals the study area as defined in §1.4.2, and is 1237 ha. The geographical area is the area bordered by the Rio Cauca in the West, the railway line in the South, the Rio Fraile and the Rio Gauchal in the East and North.

The largest part of the geographical area is used for agriculture, mainly for the cultivation of sugarcane. 150 hectares are used for industrial and residential purposes. The foreland between the Rio Cauca and the dike (26 ha) is not usable because of risk of inundation. The useful agricultural area is the total arable land (fallow land included), permanent pastures, perennial crops and gardens, including potentially effective agricultural land not yet used for agriculture is 1061 ha.

The command area is the useful agricultural area that can be irrigated by gravity, and is delimited by the highest point the water can theoretically reach. The inlet of the irrigation system is located at the end of the treatment system, near the railway bridge in the South-Western corner of the agricultural area. The elevation of the original ground level at this point is 954m. The area North of Las Dolores is hard to reach by gravity, without crossing industrial and residential areas. This is an area of 125 ha, the command area therefore becomes 936 ha. The irrigable area equals the command area.

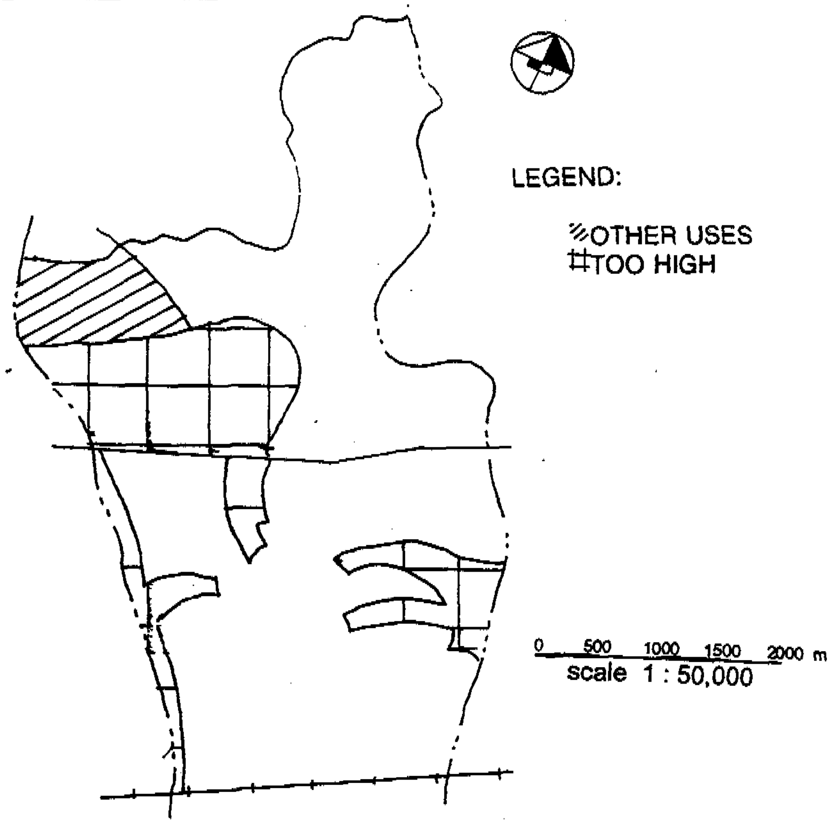


Figure 16: Geographical, command and irrigable area

5.2.2 Field Application System

The irrigation scheme employs a furrow field application system, because of its low costs, and local experience.

Modern irrigation systems can be categorized according to field application system into four categories: surface irrigation (including flood and furrow irrigation), sprinkler irrigation, trickle irrigation, and subirrigation. Flood irrigation, subirrigation and trickle irrigation are not suitable for the irrigation of sugarcane with reclaimed wastewater in the Valle de Cauca. Flood irrigation is not suitable for sugarcane irrigation, because sugarcane does not tolerate inundation (Jones et al, 1990). Subirrigation is not used in sugarcane irrigation, as the roots of sugarcane rot in satured soils. Trickle systems cannot be used with reclaimed wastewater, because the water supply must be consistently clean to prevent plugging of emitters (Smith et al., 1985).

The choice between furrow and sprinkler irrigation is a choice between a cheaper, less efficient and less flexible system and a more expensive, more efficient and more flexible system. The study area is a nearly flat area, with no major obstruction for irrigation. There is not a clear water shortage in the humid Valle de Cauca. Furrow irrigation requires a lower management security and farmers have a lot of experience with its operation. Maintenance and operation costs of both furrow and sprinkler systems depend widely on the grade of automatization. Furrow or sprinkler irrigation both have their own specific salinity problems. CENICAÑA concluded some years ago that optimizing furrow irrigation is cheaper than changing to sprinkler irrigation (Cruz, CENICAÑA, pers.comm.).

5.2.3 Combined or Separated System

The irrigation is a separated system with independent irrigation and drainage canals. A separated system is clearly arranged and easily operated. The drainage system is not designed in this study.

The selection is based on the following considerations. In a combined system irrigation canals do serve as drainage canals as well, allowing the reuse of drainage water for irrigation in lower areas. Reuse of drainage water could cause concentration of certain pollutants, which create risks for soils or crops. Separated system involves higher construction costs as more canals have to be built. Combined canals are unfeasible in gravity irrigation schemes, as the water level has to be above ground level in irrigation canals, and has to be below ground level in drainage canals. In tropical areas with heavy rainfall like the Valle de Cauca combined canals are difficult to design as they have to be able to deal with a large variation in discharges, which can cause sedimentation problems and canal damage (Brouwer, 1993)

5.2.4 Management of Scheme

The scheme is dual-managed, as the operation and management of the centralized scheme is beyond the capability of farmers. A dual-managed irrigation system is divided into a main irrigation system and a tertiary system. The main irrigation system comprises the main diversion structure, the primary and secondary canals with their structures, including the tertiary offtakes. The main system is under the control of a water committee and its general objective is to deliver water to the tertiary units. This delivery has to be with sufficient head, a reliable and flexible supply of water, in an assured way. The tertiary system, which is under the control of either one individual farmer or a number of farmers grouped in a water users association, and which includes all tertiary, quaternary irrigation canals with their structures (Ankum, 1995).

Scheme management is the coordinated approach of all parties on the implementation of the scheme objectives. Scheme management of a dual-managed irrigation scheme requires the following entities (Ankum, 1995):

1. highest authority;
2. operation and maintenance agency;
3. water users association;

The highest authority in this system is a water committee. The water committee is responsible for the treatment system as well as the irrigation scheme. This in order to guarantee a reliable supply of water for the irrigation scheme, and a certain disposition of the water for the treatment system. The water committee is headed by the head of the CVC, the regional water board. Its members are representatives of the water users, a representative of the O&M agency, and officials of EMCALI, CENICAÑA, the city council of Cali, and the city council of Palmira. The water committee is headed by somebody of the CVC to avoid tension between the city councils of Palmira and Cali.

The operation and maintenance (O&M) agency is the executive (technical) body between the highest authority and the field, and is responsible for implementing the operation an maintenance of the main irrigation and drainage schemes. The operation and maintenance agency is located at the treatment site. There is a close cooperation between the O&M of the treatment system, and the O&M of the irrigation scheme.

Water users associations manage the irrigation scheme within the tertiary units. Farmers in the Valle de Cauca are already organized in *ingenios*, and in ASOCAÑA, the national sugarcane grower society. These manners of organization should be further developed to manage the tertiary control systems, and to increase participation of farmers in the water committee.

An interesting form of farmers participation is described by Van Immerzeel (1991). He describes a peasant-to-peasant irrigation training scheme in Peru. The first part of the training consist of teaching in a particular practical situation, followed by discussion of the theory behind it. In the second part groups of farmers, representing different communities, prepare a field for irrigation. During the last two days a jury judges the different parcels. Prizes include a trophy and money, besides each participant receives a diploma and a set of tools. Participation of women is stimulated by granting them extra rewards. The best students are incorporated into the teaching program. This program is self-sustainable and has low recurrent costs, which are regained by the increase in irrigation efficiencies. The success of the program is shown by the fact that the local community continued the program, with an annual irrigation competition after the outside teachers had left.

In order to monitor the functioning of the reuse schemes, a multi-institutional environment monitoring program should be established. The program should cover, wastewater treatment and storage, irrigation, aquifers, crops, soil, geography and meteorology. Juanico (1989) describes how a monitoring program and the institute carrying it out were established in Israel, a country with a lot of experience in reuse of wastewater for irrigation.

5.2.5 Water Delivery Schedule

Water delivery is proportional: farmers get a share of the water, proportional to the surface area of thier land . An proportional delivery schedule guarantees a balance between cost and flexibility for the Cañaveralejo reuse scheme. Water-delivery scheduling refers to the frequency, rate, duration, and timeliness by which water is allocated to the various users (Brouwer, 1993; Plusquellec et al., 1994; Ankum, 1995).

Proportonal delivery guarantees farmers a large, secure, but inflexible flow of irrigation water. A constant flow of water is allocated to each farmer at all times. This garantuees a fair division of the water. In the reuse scheme water supply is constant and water demand is unpredictable. The pond system, which is the resource of the irrigation water, provides a continuous flow over the year. Yet sugarcane does accept short term water stress, and does not require a special scheduling system.

The water distribution authority maintains a constantly available supply in the primary irrigation canal, which is operated on full discharge permanently. Proportional delivery scheduling does not require a high management security, nor laborious in operation and maintenance. The costs of operating a proportional delivery system are low, as the water distribution authority does not administrate or allocate water deliveries. Proportional delivery incorporates low investment and operation costs. It does require the construction of canals to drain excess and tail-end water.

5.2.6 Operation

The structures in the irrigation scheme are fixed and no operation is necessary. This offers a reliable supply as farmers in the study area are not experienced with automatic structures.

5.2.7 Control System

The system uses upstream control, as the structures are fixed. Downstream control requires automatic structures in order to function well.

The term upstream control describes a control method that maintains a constant water level upstream of a check structure. Upstream control is usually associated with rigid top-down water delivery, but when properly operated and when allowing some tail-end losses it can provide flexible water deliveries. Upstream control is self-regulating for flood control and can be implemented with fixed, manual and automatically operated structures (Plusquellec, 1994).

5.2.8 Efficiencies

Efficiency of an irrigation scheme refers to the ratio between the total quantity of water delivered to all farms and the quantity of water actually available for plants in the rootzone. Recent evaluation reports of irrigation schemes note a substantial gap between actual and expected performance. The system of water distribution can be split into the four successive stages: water requirement of crop, field application, farm supply and project supply. The overall efficiency of a project combines the efficiencies of each stage in which water is lost. The factors that influence water losses on various levels have to be known to improve overall efficiency. (UN Economic and Social Commission for Asia and the Pacific, 1985; Plusquellec et al., 1994).

The field water application efficiency is estimated to be 60%. Field water application efficiency (e_a) is defined as the average depth of water stored in the rootzone divided by the average depth of water applied to a field. Field water application efficiencies depend on the field application system, on soil characteristics (finer texture means a higher efficiency), and on management. Literature estimates of field water application efficiencies for furrow irrigation vary between 40 and 75% (UN Economic and Social Commission for Asia and the Pacific, 1985; Asano et al., 1985; Brouwer, 1993). Present field water application efficiencies of furrow systems in the Valle de Cauca vary between 20 and 60%, the largest gain in efficiency is obtained by using alternating furrow irrigation. Although efficiency can be raised by improving the application system, it is not plausible to expect a tremendous rise if the system is managed by the same farmers: CENICAÑA expects that the maximum field application efficiency feasible with furrows, is 70%.

The distribution efficiency of the irrigation scheme is 90%, as flexitubes are used in tertiary canals. The distribution efficiency (e_d) is the ratio between the quantity of water applied from the tertiary canals and the total quantity supplied to the irrigated area, and refers to the water distribution within the tertiary unit. Distribution efficiency depends on the length and the type of canals, on operation and maintenance and on the water-delivery scheduling. In a 150 ha tertiary unit lined canals can achieve efficiencies of 95%; well maintained unlined canals with regulators can achieve 80%; and unimproved, poorly maintained watercourses only achieve a 55% efficiency (UN Economic and Social Commission for Asia and the Pacific, 1985). Distribution efficiencies in the Valle de Cauca are in the range of 50 to 90%.

The conveyance efficiency is estimated at 90% in an 1,000 ha irrigated area, and 70% at a 10,000 ha area. The conveyance efficiency (e_c) is the ratio between the total quantity of water delivered to all farm or group inlets through the water course in the area and the total quantity of water supplied into tertiary canals. Conveyance efficiency depends on the size of the irrigation scheme and the operation and maintenance of the canals. Efficiency of schemes smaller than 10,000 ha, like the Juanchito-Guanabanal-Palmaseca scheme can achieve an efficiency of 85% (UN Economic and Social Commission for Asia and the Pacific, 1985). Brouwer (1993) estimates a conveyance efficiency of 90%. A survey (Caraval and Marmolejo, 1989) showed that the conveyance efficiency in the Palmira region of both primary and secondary canals is only just over 50%. Water losses in unlined canals are hard to predict in the Juanchito-Guanabanal-Palmaseca area as soil permeability varies. CENICAÑA advises farmers to line their canals, either with plastic or with concrete, as a 95% efficiency can be reached that way.

The overall efficiency (e_p) of an irrigation project is the product of the three efficiencies mentioned above. Scarcity of water, or price increases raises awareness and are the most effective means of rising irrigation efficiencies. The overall efficiency is 49% for a 936 ha irrigation scheme (see equation 27).

$$e_p = e_a * e_d * e_c = 60\% * 90\% * 90\% = 49\% \qquad \textbf{(27)}$$

5.3 Irrigation Water Requirements

5.3.1 Water Requirement

The water requirement is the volume of water applied per unit area of land per unit time. The water requirement depends on the climate, the crop, the quality of the irrigation water, and the field application method. Peak water demands can be reduced by rotating planting dates, and storage of irrigation water. The leaching requirement and the preplant irrigation have to be known to calculate the water requirement.

Leaching Requirements

The leaching requirement is that portion of the applied irrigation water entering the surface that has to percolate below the rootzone in order to leach salts out of the rootzone (see equation 28). When analyzing the salt balance presented in §2.3.2, the leaching fraction was found to be the quotient of the flow of irrigation water and the flow of drainage water, as well as the quotient of the salinity of the irrigation water and the salinity of the drainage water (equation 29).

$$LR = LF * V_f \tag{28}$$

where: LR = leaching requirement [mm/month];
LF = leaching requirement [-];
V_f = field water requirement [mm/year];

$$LF = \frac{D_{dw}}{D_{iw}} = \frac{EC_{iw}}{EC_{dw}} \tag{29}$$

where: D_{iw} = flow of irrigation water [m^3];
D_{dw} = flow of drainage water [m^3];
EC_{iw} = electric conductivity of irrigation water [dS/m];
EC_{dw} = electric conductivity of drainage water [dS/m].

The average electric conductivity (EC_{iw}) of the irrigation water is 0.7 dS/m. The salinity tolerance of a crop (EC_e) refers to the electric conductivity of the soil in the rootzone, The salinity tolerance is in between the electric conductivity of the irrigation water and the electric conductivity of the drainage water. Sugarcane crop tolerance is 1.7 dS/m. Rhoades and Loveday (1990) state the electric conductivity of the soil in the rootzone (EC_s) can be estimated to be the intermediate between EC_{iw} and EC_{dw}. Vakgroep Gezondheidstechniek en Waterbeheersing (1993) uses a similar approach (equation 30).

$$EC_s = EC_e = 0.5 * (EC_{iw} + EC_{dw}) \tag{30}$$

where: EC_s = electric conductivity of the root zone [dS/m];
EC_e = tolerated electric conductivity of crop [dS/m].

Combining equations 29 and 30 results in the following relationship for the leaching fraction:

$$LF = \frac{EC_{iw}}{2 * EC_e - EC_{iw}} \tag{31}$$

Ayers and Westcot (1985) suggest another relationship for the leaching fraction, based on the idea that the water uptake of the crop is not linear, and therefore the soil water salinity is not the intermediate between EC_{iw} and EC_{dw} (equation 32).

$$LF = \frac{EC_{iw}}{5 * EC_e - EC_{iw}} \tag{32}$$

According to the Rhoades and Loveday relationship (equation 31) the leaching fraction is 0.26, which means that 26% of the irrigation water should percolate beyond the rootzone. The Ayers and Westcot relationship (equation 32) results in a leaching fraction at 0.08.

The usual inefficiencies of water application satisfy this leaching requirement. The field water application efficiency (e_a) of this irrigation system is 0.6, this means an extra 63% of the crop requirement is applied to account for losses, part of which is deep percolation. Furthermore rainfall applies a large quantity of low-salinity water. It can be safely assumed that this percolation can satisfy the leaching fraction of 26%, calculated above, so no extra leaching requirement has to be accounted for (Ayers and Westcot, 1985).

Preplant irrigation
Preplant irrigation is not practiced in sugarcane cultivation in the Valle de Cauca.

Field Water Requirement
The field water requirement is calculated using equation 33 (Brouwer, 1993). Open pan evaporation is 1627 mm for an average year (Granja ICA station), average rainfall is 1033 mm (Planta Rio Cauca station), of which 80% (is 827 mm) is the effective precipitation. The average crop coefficient for a complete grow cycle is 0.42 (see §5.1.3 and §5.3.2). Preplant irrigation and the leaching requirement are both 0. The field water application efficiency is 0.6. The water requirement for several typical years are presented in table 18. This table shows that in wet years hardly any water has to be irrigated. Further monthly rates are discussed in the next paragraph on the capacity line, which is based on a 80% dry year.

$$V_f = \frac{E_0 * k_c - P_{eff} + V_t + LR}{e_a} \tag{33}$$

where
E_0 = evaporation [mm/year];
k_c = crop coefficient [-];
P_{eff} = effective precipitation [mm/year];
V_t = volume required for preplant irrigation [mm/year];

Nitrogen Leaching Control
The design of the reuse system should avoid nitrogen percolating into potable groundwater aquifers. The procedure to estimate the percolation of nitrogen is based on a procedure presented in the EPA Process Design Manual for Land Treatment of Municipal Wastewater (USEPA, 1977) and presented in Smith et al. (1985). Nitrogen concentration in the percolate should not exceed 10 mg/l annually. The nitrogen uptake by crop is 10 g/m^2 per year for a first crop and higher for ratoon crops (see table 16, §1.1.3). The fraction f can be estimated to be 0.2. In table 18 the maximum irrigated volume for nitrogen control are calculated for primary effluent (26 mg N/l) and pond effluent (15 mg N/l). The table shows that nitrogen percolation problems do not occur, as the water requirement is always lower than the maximum irrigated volume for nitrogen control.

$$V_n = \frac{C_{dw}*(P_{eff}-E_o*k_c) + U}{(1 - f)*C_{iw}-C_{dw}} \tag{34}$$

where: V_n = maximum irrigated volume for nitrogen control [m/year];
C_{dw} = nitrogen concentration in drainage water [mg/l];
U = nitrogen uptake by crop [g/m^2 * year];
C_{iw} = nitrogen concentration of irrigation water [mg/l];
f = fraction of applied nitrogen removal by denitrification and volatilization [-].

Table 18: Annual irrigation gifts

year	effective precipitation	evapo-transpiration	water requirement	maximum irrigated volume	
				primary effluent	pond effluent
	mm/year	mm/year	mm/year	mm/year	mm/year
0.2 dry year	1155	688	25	1358	7331
0.4 dry year	847	688	164	1073	5793
0.6 dry year	626	688	375	868	4686
0.8 dry year	390	688	649	650	3508
average	754	688	304	987	5329

5.3.2 Peak Reduction: Rotation and Storage

Rotation
Rotation of the planting date of the sugarcane reduces peak water demand, and optimizes use of equipment and man power. Sugarcane can be planted in every month of the year in the Valle de Cauca. Crop coefficients in an area planted at the same date vary over the growing age (figure 17a). Crops are planted in different seasons in larger areas. In a large area the crop is planted in every month of the year and the monthly crop coefficient equals out to 0.42 (figure 17d).

Storage
Storage reservoirs in irrigation schemes regulate the available water in the most useful way, by meeting peak irrigation demands in excess of the average wastewater flow. The size and location of reservoirs depend on their function. Storage of wastewater for irrigation minimizes disruption in the operation of the treatment system and the irrigation scheme. It equalizes daily variations in flow; provides insurance against the possibility of unsuited reclaimed wastewater entering the irrigation scheme; and provides additional treatment (Asano et al., 1985).

Night reservoirs are included in the design to save the water flow at night for irrigation during the day. This kind of reservoirs is much smaller than the kind for meeting peak demands in dry seasons, but still can provide large water savings. The reservoirs are located at the top end of the irrigation system as water can still be distributed to every tertiary unit. and head is available at the top end of the canal. The night reservoirs are designed to store the treatment discharge during 12 hours at night. Farmers do not irrigate on Sunday. Storage on Sundays requires large reservoirs, which induce large investments but small water savings.

Reservoirs for meeting peak irrigation demands in dry seasons are expensive to construct and hard to operate in the unpredictable climate in the Valle de Cauca. This type of reservoirs is therefore not included in the design.

In this reuse scheme, where only part of the effluent of the Cañaveralejo plant is reused, the wastewater flow does not fluctuate. The retention time of the pond system provides insurance against the possibility of unsuited reclaimed wastewater entering the irrigation scheme.

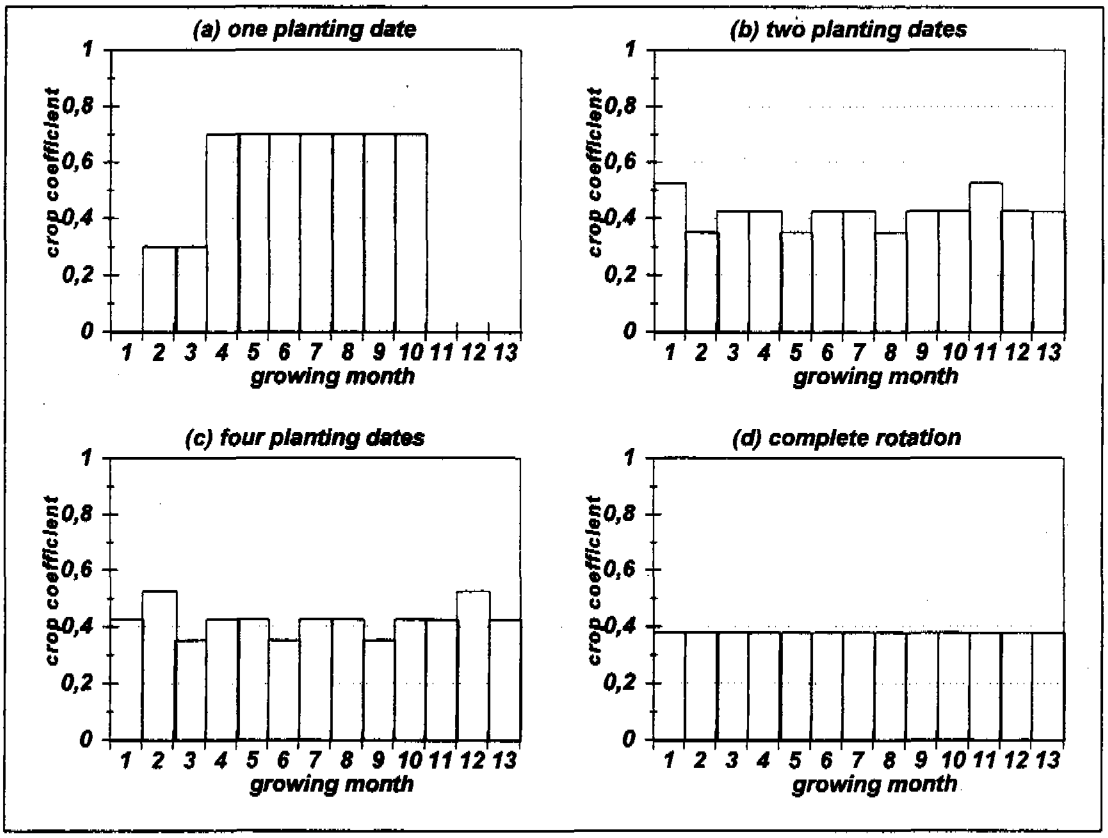


Figure 17: crop coefficients with rotation

5.3.3 Capacity Line

The capacity line (figure 18) estimates the water requirement per hectare necessary for a certain area of land taking into account peak reduction by crop rotation and losses in the distribution and conveyance system. In a large area the average water requirement decreases due to crop rotation, but increases due to operational and leakage losses. The discharge necessary to irrigate a certain area can be determined by multiplying the discharge per hectare by the amount of hectares.

The capacity line in figure 18 shows the discharge needed to irrigate an area of 936 ha is 384 l/s, 24 hours per day, 7 days per week. A surface area of 13,300 hectares could be irrigated with the available discharge of the Cañaveralejo plant (7.6 m^3/s).

On Sundays farmers do not irrigate: for 24 hours water is neither used nor stored. The discharge of the pre-irrigation treatment in the remaining time must be 450 l/s in order to meet peak water demands (see equation 35). This is the design effluent of the pre-irrigation treatment designed in chapter 4.

$$Q_{tr} = \frac{7}{6} * Q_{con} = \frac{7}{6} * 384 = 450 \ l/s \qquad \textbf{(35)}$$

where: Q_e = effluent discharge of treatment system [l/s];
Q_{con} = discharge with continuous delivery [l/s];

The capacity of the irrigation canals is designed taking into account irrigation hours and storage of water: instead of a continuous flow, there will only be a flow during 12 hours a day from Monday till Friday. The capacity line is altered in order to serve as a base for canal design. The discharge to irrigate 936 ha in 72 hours, is 900 l/s (see equation 36).

$$Q_{int} = \frac{7*24}{6*12} * Q_{con} = \frac{168}{72} * 384 = 900 \ l/s \qquad \textbf{(36)}$$

where: Q_{int} =discharge with intermittent flow [l/s];

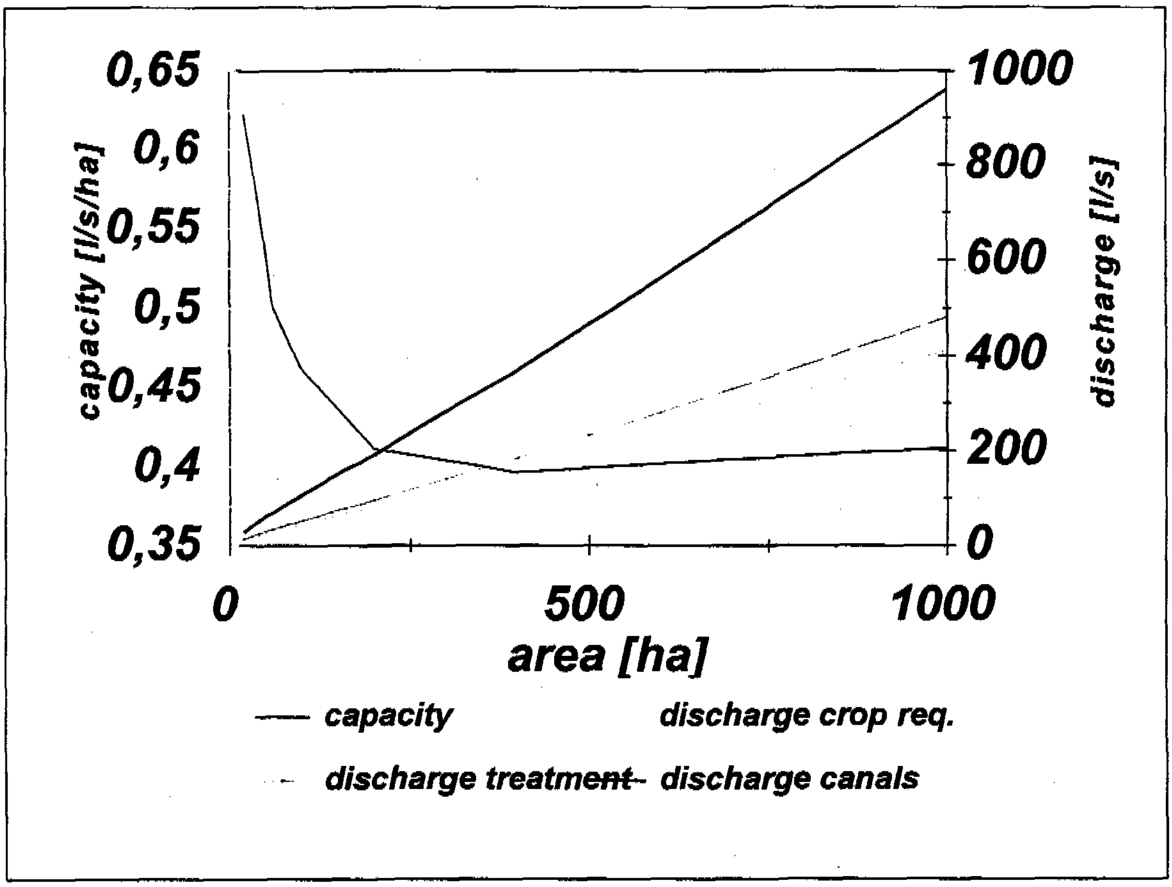


Figure 18: Capacity line

5.4 Layout of Irrigated Field Area

5.4.1 Grouping in Tertiary Units

The layout of the tertiary units is based on hydrological considerations, as the explicit microrelief in the study area would cause a very complex and expensive per-farm distribution system. The study area is divided into 10 tertiary units, which cover between 35 and 157 ha (see table 19). A tertiary unit is defined as that part of a dual-managed irrigation scheme that is managed by the water users association and not by the O&M agency. Water is delivered proportionally to tertiary units at a central inlet, within the tertiary unit farmers manage the water.

The study area is divided into several dales draining either to the Rio Fraile or to the Rio Cauca. Farms often cross ridges and dales. A per-farm distribution system would imply an extensive main irrigation system and would not allow gravity flow in all irrigation ditches (see figure 27, appendix V). Tertiary units based on hydrological considerations ease the construction of the main irrigation system. A hydrology-based layout of the irrigated field area has the disadvantage that farmers have to share water within tertiary units, which complicates the management of the scheme and decreases efficiencies. As most farmland is rented to the *ingenios*, which cultivate vast areas of sugarcane, this problem is overcome.

Table 19: Tertiary units

unit	area [ha]	discharge [l/s]
O1	100	98
O2	138	130
03	157	142
O4	156	141
E1	109	105
E2	47	49
E3	68	69
E4	57	59
E5	69	70
E6	35	37

5.4.2 Layout of Distribution System

The primary canal of the irrigation system follows the central rim situated North-South in the study area. The primary canal is 5050 m long, and serves the ten offtakes to tertiary units (see figure 20).

5.5 Design of Irrigation System

The design of the irrigation system is an iterative process between the design of reservoirs, the design of canals, and the design of structures.

5.5.1 Design of Reservoirs

In a pond system the final pond can function as a reservoir by varying the water level. Primary effluent cannot be stored without prior treatment.

The volume of a reservoir with a 12 hours storage capacity is 20,000 m^3 (equation 37).

$$V_r = 12 * 3600 * \frac{Q_e}{1000} = 12 * 3600 * \frac{450}{1000} = 20{,}000\ m^3 \tag{37}$$

where: V_r = Volume of the reservoir [m^3];
Q_e = effluent discharge of treatment system [l/s];

In a duckweed pond system the storage capacity can be provided by varying the water level in the final pond. The final pond designed in chapter 4 is 4.5 ha. An extra water depth of 0.45 m should be allowed for to store a volume of 20,000 m^3.

Primary effluent requires additional treatment prior to storage, to prevent reservoirs from becoming septic (Smith et al., 1985). Night reservoirs cannot be used. The discharge of the influent must be intermittent, which means that at night and in weekends no water is transported across the river.

5.5.2 Canal Design

The primary canal delivers water to the tertiary offtakes. The canal has to deliver water to tertiary units proportionally to the surface area of the unit.

The requirements for the primary canal are:

1. The capacity at the head end has to be 900 l/s and diminishes along the canal;
2. Freeboard of all canals is 0.20 m;
3. The water level at the top end must be below 954.53 m+, in order to allow free flow over the outlet structures of the waste stabilization system;
4. The inlets of the tertiary units must be served to allow for free flow till the field in the complete tertiary unit;
5. The water level upstream of the structures must allow for free flow over the structure and in the ongoing canal;
6. Net excavation must be kept to a minimum;

The hydraulic radius in a trapezoidal canal depends on the width of the canal, the slope of the sides, and the depth of the canal, and is given by equation 38. Dimensions of open canals are calculated with the formula of Strickler (equation 39), and presented in table 21, 22. The roughness coefficient is 75 $m^{1/3}/s$, the side slope of the canals is 1:1, the ratio between depth and width is 1.5.

$$R = \frac{A}{P_{can}} = \frac{b*y + n*y^2}{b + 2*y*\sqrt{(1+n^2)}} \tag{38}$$

where: R = hydraulic radius [m];
A = surface area [m^2];
P_{can} = wetted perimeter [m];
b = bed width [m];
y = water depth [m];
n = side slope [-];

$$Q = k_s * A * R^{\frac{2}{3}} * s^{\frac{1}{2}} \tag{39}$$

where: Q = discharge [m^3/s];
k_s = Strickler roughness coefficient [$m^{1/3}/s$];
s = slope [-];

Table 20: Canal dimensions

CANAL	Q	length	water depth	canal depth	bottom width	s
	l/s	m	m		m	10^{-3}
WSP-O1	900	0	-	-	-	-
O1 - O2/E1	802	900	0.64	0.84	0.96	0.5
O2/E1 - E2	567	800	0.56	0.76	0.85	0.4
E2 - E3	518	1100	0.55	0.75	0.82	0.4
E3 - E4	449	700	0.52	0.72	0.78	0.4
E4 - E5	390	tube	0.49	0.69	0.74	0.4
E5 - O3	320	850	0.46	0.66	0.68	0.4
O3 - O4/E6	178	700	0.35	0.55	0.53	0.5

5.5.3 Structure Design

The offtakes consist of two sharp-crested weirs to implement the proportional delivery schedule. The easiest way to obtain proportional water delivery is the combination of two weirs (Bos, 1978; Brouwer, 1993). Weirs are robust, cheap and easy to operate. The weirs in the offtake and the ongoing canals have the same height. The length of the weirs should be proportional to the discharge which has to flow over them.

The cost of construction, operation, and maintenance is an important criterion in the selection of structures. The ease with which a discharge can be measured or regulated, reduces the cost of operation. The selection of the type and the shape of structures is influenced by the available head and the required head at the discharge measuring site. Ease of operation saves labor and ensures more efficient distribution of water over the irrigation area. Simplification of structure design is desirable, but it should not come at the expense of the quality of irrigation services, which is increased by adjustable structures (Bos, 1978; Plusquellec, 1994). Robustness of design denotes the capacity of structures or equipment to perform under adverse conditions and is a very important feature under real world conditions (Plusquellec, 1994).

The drop in energy level over the weir should be kept to a minimum, as the available head in the flat irrigation area is limited. The tailwater level should be lower than 5/6 H_c to satisfy the free flow condition (equation 40; Brouwer, 1993). The weirs in the ongoing canal, the water-level regulators, have to be duckbill weirs in order to limit the drop in energy level over the weir in the main system. Duckbill weirs are fixed weirs with a long crest which is folded in the shape of a duckbill into a canal. Duckbill weirs reduce fluctuations of the upstream water levels within relatively narrow limits, are robust and simple to construct.

$$Q_{max} = c * b_w * H_c^{\frac{3}{2}} \tag{40}$$

where: b_w = width of weir [m];
Q_{max} = design discharge [m^3/s];
H_c = upstream energy level above crest [m];
c = weir coefficient for free flow [$m^{1/2}/s$] ≈ 1.9;

The sizes of the offtake structures are summarized in table 22.

Table 21: Offtake dimensions

OFFTAKE	upstream water depth	height weir	H_{crest}	ongoing canal		offtake	
				$Q_{offtake}$	length weir	$Q_{offtake}$	length weir
	m	m	m	l/s	m	l/s	m
O1	0.64	0.49	0.15	802	7.27	98	0.89
O2	0.64	0.49	0.15	567	5.14	130	1.18
03	0.46	0.31	0.15	178	1.61	142	1.29
04	0.35	0.20	0.15	178	1.61	141	1.28
E1	0.64	0.59	0.15	567	5.14	105	0.95
E2	0.56	0.41	0.15	518	4.69	49	0.44
E3	0.55	0.40	0.15	449	4.07	69	0.63
E4	0.52	0.37	0.15	390	3.53	59	0.53
E5	0.49	0.34	0.15	320	2.90	70	0.63
E6	0.35	0.20	0.15	178	1.61	37	0.34

5.6 Example of Design of Tertiary Unit

A tertiary unit is that part of a dual-managed irrigation scheme that is managed by the water users association (Ankum, 1995). In order to provide an example of how a tertiary unit could be designed, the tertiary unit O3, which is located in the Northern part of the study area is designed. The design method of the main irrigation scheme is applied to a smaller area: §5.6.1 discusses the layout within the tertiary unit, in §5.6.2 the canal system is designed, and §5.6.3 treats of the control system.

5.6.1 Layout within the Tertiary Unit

A tertiary unit is divided into quaternary units, the area served by the quaternary canal. The size of the quaternary units depends on the amount of furrows served by the canal, the furrow spacing, and the length of the furrows.

The furrows can be spaced further apart, instead of alternating furrows (see figure 21). Enlarging the distance between furrows increases the amount of plants, and thus the yield per hectare. The extra yield can be estimated by comparing the average spacing of sugarcane plants: this is 1.5m by an alternating furrow system and 3.5/3=1.17m in new furrow system. The yield increases can be estimated as 20%, as the rows take up 22% less space (Cruz, CENICAÑA, pers.comm.).

The uniformity of application and the field application efficiency can be increased by decreasing the furrow length. On the other hand reduced furrow length creates a need for more quaternary canals and therefore increases construction, operation, and maintenance costs. The maximum length of the furrows in this scheme is 100 m.

The length of the quaternary canal is based on the topography of the area. Longer quaternary canals require less diversion structures and shorter tertiary canals. Present maximum canal length is 800 m. The maximum canal length in the design is 1600 m, because of the larger furrow spacing and resulting reduced number of furrows.

The minimum size of a quaternary unit is based on a unit flow of 60 l/s which a *regalador*, an irrigation worker, can handle properly. The discharge is 60 l/s, which feeds 20 furrows with 3-4 l/s for 1 to 2 hours at the same time. The minimum size of a quaternary unit is $20*100*3.5 = 7000\ m^2 = 0.7$ ha..

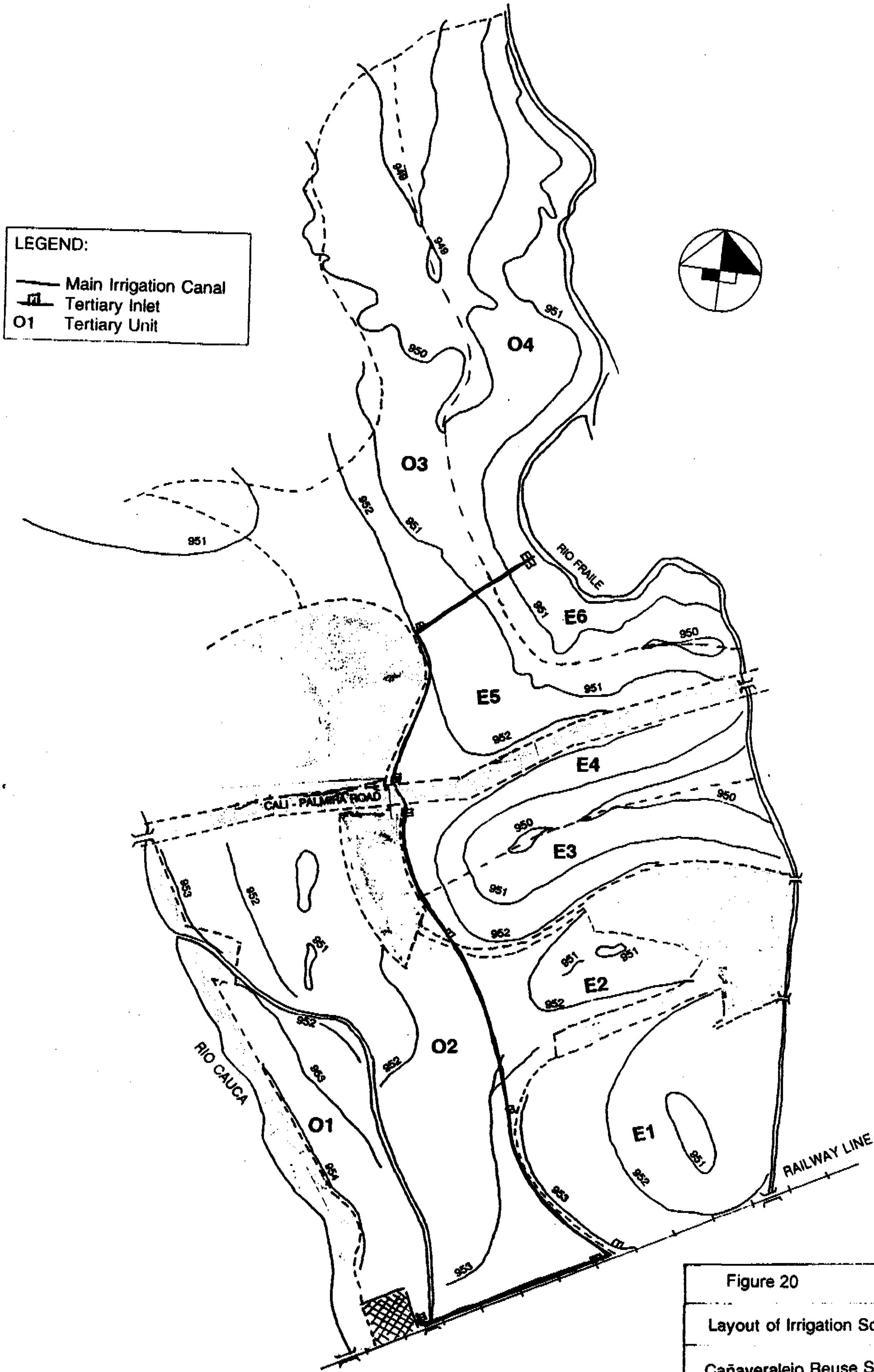


Figure 20

Layout of Irrigation Scheme

Cañaveralejo Reuse Scheme

Meike van Ginneken

Date: 8 - 96

Scale 1 : 15,000

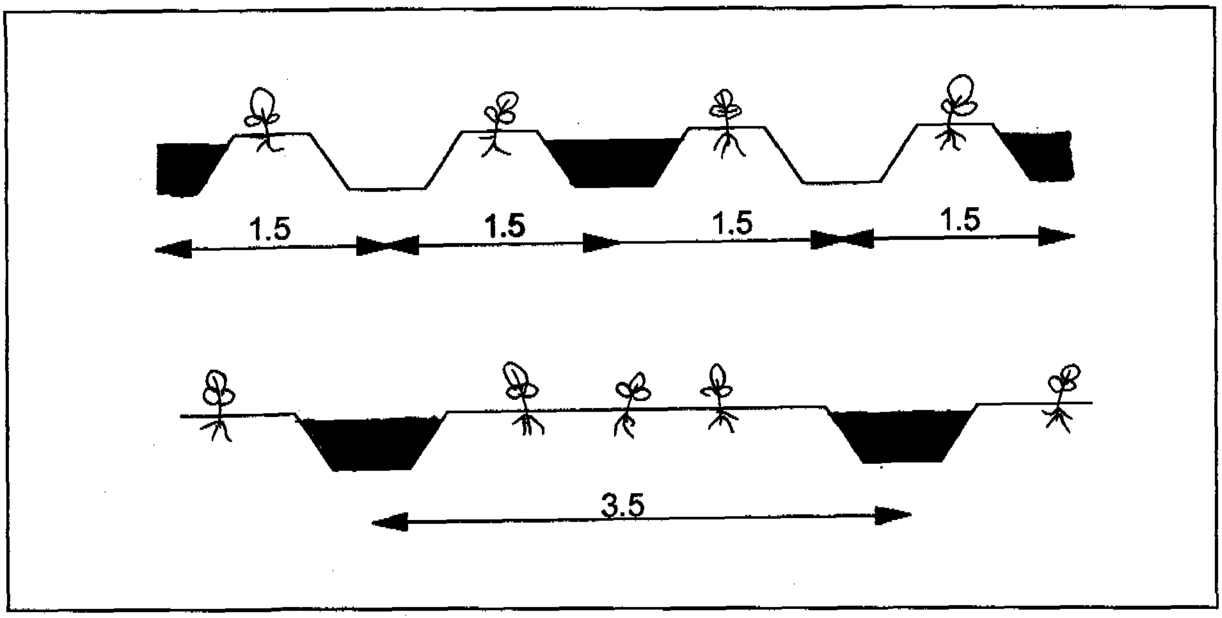


Figure 21: furrow spacing

5.6.2 Canals

Tertiary Canals: Open Lined Canals
The tertiary canals are open lined canals. The canals are designed using the Strickler formula (equation 39). Minimum slope is 2 ‰. Freeboard is 0.20 m. The O3 tertiary canal has a slope of 2 ‰, a depth of 0.42 m and a width of 0.62 m.

Quaternary Canals: Flexitubes
Flexitubes are flexible polyethylene tubes, with openings at regular spacings along the pipeline. The spacing of these openings correspond with the furrow spacing. The benefits of the use of flexitubes are (1) water savings by increasing efficiency, (2) labor savings, (3) increase of the effective crop area, and (4) possibilities for mechanization of yield (Cruz et al., 1995). Smith (1990) states that outflow along the pipe (and so the variation in the inflows to each furrow) varies up to 40% from the mean average. Benitez (1994) found that the uniformity of application with flexitubes is better than with open tertiary canals.

The Strickler formula for tubes (with R = 1/4*D and A = 1/4*π*D²) is given in equation 41. Flexitubes need a fairly constant slope to function properly. A cautious roughness coefficient estimate for polyethylene is 91 $m^{1/3}/s$. A flexitube with a slope of 2‰, and a discharge of 60 l/s, should have a diameter of 0.32 m.

$$Q = 0.312 * k_s * D^{2\frac{2}{3}} * s^{\frac{1}{2}} \tag{41}$$

5.6.3 Tertiary Control System
Structures within the tertiary unit are operated and maintained by the water users association, and should be simple and robust. The tertiary control system consist of water-level regulators in the tertiary ongoing canal, in combination with offtake structures in the quaternary canals (flexitubes).

The inlet of the flexitubes can easily be provided by a piece of steel piping perforating the side of the canal, which is provided with an on/off gate. The top of these orifices is placed well below the upstream water level. The head loss at the inlet can be calculated with the formula for local head losses, with μ=0.82 (King et al, 1990).

$$dH = C_l * \frac{v^2}{2*g} = (\frac{1}{\mu}-1)^2 * \frac{v^2}{2*g} \qquad \textbf{(42)}$$

where: H_l = local head loss [m];
k = local head loss coefficient [-]
v = velocity [m/s];
g = 9.81 m^2/s;
μ = contraction coefficient [-];

A water-level regulator has to be installed downstream of the inlet to keep up the water level in front of the orifices. Broad-crested weirs are selected as they are easy to install, robust and cheap.

5.7 Cost Estimate

Total costs include investment costs (land acquisition, construction of canals and offtakes, establishment of O&M organization), and annual costs (O&M costs, pumping of water supply, fertilization). The total costs for the present ("old") irrigation scheme, and the reuse ("new") scheme are presented. The costs given below are rough estimates. Only the acquisition of land and the contracting costs are considered. Appendix IX lists individual prices of parts of the scheme.

5.7.1 Investment in New Irrigation Scheme

Investment costs of the new irrigation scheme include the construction of canals and offtakes, and the costs for establishing the organization. The investments for the irrigation scheme include investments paid by the water delivery organization (for the main system), and investments paid by the farmers (for tertiary system). The old irrigation scheme does not involve investment costs. The investment costs for the new scheme are shown in table 22.

Table 22: Investment costs of new irrigation scheme

item	unit	unit cost	amount	cost
main system				
land acquisition	m^2	$10	50,000	$500,000
main canal	m	$36	5,050	$180,000
offtake	#	$7,700	10	$77,000
tertiary system				
canals	m	$18	15,600	$277,000
offtakes	#	$10	200	$2,000
flexitubes	m	$2	62,400	$130,000
TOTAL				**$1,200,000**

Main System
The investment costs for the main system are US$800,000. Costs include land acquisition, costs of construction of canals, and costs of construction of offtakes.

The cost of purchasing 5 ha of land is US$500,000. These five hectares of land must be purchased for the construction of the main irrigation system. This figure is based on a strip of 10 m wide for the 5050 m long primary canal.

The costs of construction the main canal is US$36 per meter length (see table 23). The cost of construction per unit length of the primary canal are based on the cross-sections presented in figure 20 and table 21, 22. The costs consists of excavation, concrete for lining, and labor for lining. CENICAÑA estimates the costs of construction of primary canals at US$17 per meter (Cruz, CENICAÑA, pers.comm.).

Table 23: Costs of primary canal per m length

item	unit	unit cost	amount	cost
excavation	m^3	$3.44	3	$10
concrete (1500 psi)	m^3	$81	0.3	$24
labor	day	$5	0.2	$1
TOTAL				$36

The costs of construction of an offtake are US$7,700 (see table 24). These costs are based on dimensions of the offtake O3 (see figure 23). Offtake 03 is a fairly large offtake, it is presumed that other offtakes have the same costs.

Table 24: Costs of primary canal per m length

item	unit	unit cost	amount	cost
excavation	m^3	$3.44	5	$17
concrete (2000 psi)	m^3	$82	3	$250
reinforcement ⌀16	m	$0.5	100	$50
formwork	m^2	$1	25	$25
distributors	m	$8,000	0.9	$7,200
labor	day	$5.40	25	$135
TOTAL				$7,700

Tertiary System
The total investment for farmers to implement the new scheme are US$400,000. These costs include the construction of canals, and offtakes, and the purchase of flexitubes.

Secondary canals are two to four times smaller than the primary canal. Construction costs are estimated at US$18, which is half of the costs of the primary canal. Seventeen meters of secondary canal is constructed per hectare.

Quaternary offtakes consist of a piece of steel piping in the site of the canal, which is supplied with an on/off gate. The costs of an offtake are $10, (US$4 for material, and US$6 for labor; Benitez, 1994). Flexitubes cost US$2 per meter; 67 m of flexitube is provided to irrigate one hectare.

5.7.2 Annual Costs

The annual costs of the new irrigation scheme (US$1,250,000) equal those of the old scheme. Operation and maintenance costs are higher for the new scheme. Costs of water supply and fertilization are lower for the new scheme. The other categories of costs for the old and the new irrigation are presumed to be the same. The costs for these categories are based on unpublished figures from CENICAÑA. These categories include: drainage; clearing and grubbing; soil preparation; weed control; and miscellaneous. The annual costs are presented in table 25.

Table 25: Annual costs of old irrigation scheme

item	unit	old irrigation scheme		new irrigation scheme	
		unit cost	annual cost	unit cost	annual cost
operation and maintenance			$200,000		$390,000
water supply	ha/year	$110	$100,000	$0	$0
fertilization	ha/year	$150	$140,000	$123	$110,000*
drainage	ha/year	$106	$100,000	$106	$100,000
clearing and grubbing	ha/year	$493	$460,000	$460	$460,000
sowing	ha/5 years	$383	$70,000	$383	$70,000
weed control	ha/ year	$60	$60,000	$60	$60,000
harvesting	ha/year	$35	$40,000	$35	$40,000
miscellaneous	ha/year	$95	$90,000	$95	$90,000
TOTAL			$1,250,000		$1,250,000

* $90,000 for reuse scheme with primary effluent

Operation and Maintenance

The annual costs of a centralized irrigation scheme using reclaimed wastewater include the costs for the operation and maintenance center, the actual operation of the main system, and the maintenance of canals and offtakes. The costs of the operation and maintenance center are the sum of two salaries: one supervisor and one laborer. The costs for O&M of the new system are based on the presumption that operation and maintenance takes twice as much time in the more elaborate new canal system.

Water Supply

The costs of water supply for the old system US$100,000 for $2.9 * 10^6$ m^3 water. Presumptions on which this estimate is based, are: (1) 20% of this water is derived from groundwater and 80% from surface water; (2) an average overall efficiency of 30%; and (3) a crop coefficient of 0.42 after rotation. The pumped discharge per year is $2.9*10^6$ m^3. This is the average water requirement (304 mm, see table 18), times the surface area (936 ha).

CENICAÑA estimates the cost of pumping of groundwater from wells to be US$0.07 per m^3 in February 1996. Ingenio Mayagüez, a sugarcane factory that cultivates several farms in the survey area states a price of US$0.084 in March 1996. The CVC established taxes on the use of groundwater in 1995, at present they range from 0.07 to 0.1 dollarcents per m^3 (CVC, 1995; Cruz, pers.comm.; Medina, pers.comm).

Table 26: Operation and Maintenance Costs

item	unit	unit cost	old irrigation scheme			new irrigation scheme		
			amount per ha	cost per ha	annual cost	amount per ha	cost per ha	annual cost
O&M center					$0		costs scheme	$10,000
operation	h	$0.68	40	$27	$25,000	80	$54	$51,000
maintenance								
manual	h	$0.68	33	$22	$21,000	66	$45	$42,000
mechanized	h	$18	6	$111	$104,000	12	$221	$207,000
materials	h	$1.29	4.2	$5	$5,000	8.4	$11	$10,000
reservoirs	h	$4	4.2	$16	$5,000	8.4	$0	$0
chemical	h	#4.6	4.2	$17	$15,000	8.4		$2,000
supervision	h		4.2	$19	$18,000	8.4	$39	$36,000
TOTAL					$200,000			$390,000

CENICAÑA estimates the cost of pumping of surface water US$ 0.02- 0.025 per m^3 in February 1996. Ingenio Mayagüez states a price of US$ 0.037 per m^3 for surface water. Taxes on the use of surface water were set in July 1995 and vary between 0.15 and 0.5 dollarcents per m^3 (CVC, 1995; Cruz, pers.comm.; Medina, pers.comm).

Fertilization

Less fertilizers have to be applied to the crop, when using nutrient rich wastewater for irrigation. The amount of a nutrient provided by the wastewater can be estimated by multiplying the nitrogen concentration by the amount of water applied per hectare per crop period. Table 27 shows the concentration of nutrients in the waters, the amount of nutrients provided by the water, and the amount of fertilizer which has to be applied.

The application of reclaimed wastewater saves fertilizers, labor, and equipment. Table 28 shows the savings obtained by the reuse system.

Table 27: Nutrients gifts of wastewater

			pond effluent	primary effluent	ground/surface water
Nitrogen	conc. irrigation water	mg/l	15	26	0
	gift from irrigation water	kg/ha	46	79	0
	need from fertilizer	kg/ha	94	61	140
	Urea (46% N)	kg/ha	205	133	304
Phosphorus	conc. irrigation water	mg/l	2	3	0
	gift from irrigation water	kg/ha	6	9	0
	need from fertilizer	kg/ha	34	31	40
	SFT (46% P)	kg/ha	74	67	87

Table 28: Fertilization costs

item	unit	unit cost	ground/surface water		pond water		primary effluent	
			amount	annual cost	amount	annual cost	amount	annual cost
urea	kg	$0.27	304	$77,600	205	$52,300	133	$34,000
SFT	kg	$0.28	87	$23,100	74	$20,000	67	$17,800
KCl	kg	$0.28	40	$10,600	40	$10,600	40	$10,600
manual application	day	$5	0.8	$3,900	0.53	$2,600	0.4	$1,900
mechanized application	day	$17	0.9	$14,000	0.6	$9,300	0.45	$7,000
soil sampling	#	$10	1	$9,600	2	$19,200	2	$19,200
maintenance	day	#5	0.4	$2,000	0.4	$2,000	0.4	$2,000
TOTAL				**$140,000**		**$110,000**		**$90,000**

5.8 Benefit Estimate

The benefit of the irrigation scheme is the yield of sugarcane of the irrigated area. The value of the present yield is US$2,200,000. The value of the yield in a new irrigation scheme is US$2,700,000. The price paid for raw sugarcane at the end of 1994 was US$19 per ton (CONE, 1995). The average yield of raw sugarcane in the municipality of Palmira is 125 ton/ha per year (CONE, 1995).

The yield in the new system (US$2,700,000) is 21% higher than the yield in the old system. This increases is caused by the reduction of the number of furrows, and the use of furrows. The new furrows take up 22% less space, so the yield increase is estimated to be 20%. Reduction of the amount of furrows increases the number of plants per hectare and therefore the yield per hectare. The use of flexitubes as quaternary canals, increases the actual yielded area. The useful length of the furrow is 2 m longer, which increases the used agricultural area from 98.3% to 99.8%, this is a relative increase of 1.2% of the yield per ha (Benitez, 1994).

$$Y_{new} = 1.2 * 1.012 * 125\ ton/ha * 19\ US\$/ton = 2884\ US\$/ha \tag{43}$$

where: Y_{new} = yield with improved irrigation scheme [US$/ha];

6 Feasibility Estimate of "Cañaveralejo" Project

This chapter shows alternative ways to solve the sanitary and agricultural problems; it does not aim at determining the best solution. The judgment of the feasibility of two reuse alternatives is based on a comparison with a reuse alternative. This comparison uses a set of criteria which regards all results of the implementation of a reuse project. The alternatives which are elvaualted are discussed in §6.1. The second paragraph (§6.2) discusses the financial cost-benefit analysis. The checklist in §6.3 uses the criteria set generated in chapter 3. In the final paragraph (§6.4) the evaluation is discussed.

The feasibility of a reuse project depends on two values: the "sanitary" value and the "agricultural" value. The "sanitary" is the value of diminishing (treatment to prevent) contamination of natural resources, and can be estimated by the value of alternative treatment systems. The "agricultural" is the value of providing an alternative water resource for agriculture and can be estimated by the price of existing resources.

6.1 Alternatives: Combinations of Treatment and Irrigation

Four possible treatment systems are mentioned in chapter 4, appendix VI, and appendix VIII: a pond system, a subsurface flow wetland system, only primary treatment, and an activated sludge plant. In chapter 5 the old unimproved irrigation scheme is described, and a reuse irrigation scheme is designed. Four treatment options and two irrigation options result in eight alternatives (see table 29).

Only three alternatives are discussed to increase transparency. The alternatives show the distinctions between the various treatment options. The feasibility to reuse wastewater can be judged by the feasibility of the two reuse alternatives. The wetland alternatives are not looked at, as they are more costly than pond alternatives.

Costs and benefits of the following situations are summarized:

1. Reuse with pre-irrigation treatment pond system; The pond system which is designed in chapter 4 is combined with the irrigation scheme design of chapter 5.
2. Reuse with only primary pre-irrigation treatment; The possibility to reuse the primary effluent without further treatment for irrigation is mentioned in chapter 4. This alternative results in lower treatment costs, but higher health and agricultural risks. The alternative is discussed here to show the price of reducing risks.
3. No reuse, wastewater treatment with activated sludge and sludge digester, unimproved irrigation scheme; Secondary treatment with an activated sludge process will be provided in the Cañaveralejo plant. The existing plans of EMCALI are the base for the treatment part of this alternative. The costs and benefits of the irrigation scheme are based on present practices.

Table 29: alternatives

	activated sludge plant	pond system	wetland system	primary treatment
old irrigation scheme	alternative 3			
reuse irrigation scheme		alternative 1		alternative 2

6.2 Financial Cost-Benefit Analysis

The costs and benefits of the reuse scheme are fairly hard to estimate, as there is a scarcity of economic data on large-scale reuse systems. Most of the economic data in the literature refer either to experimental or to hypothetical reuse systems (Edwards, 1992).

The net present value method is used to account for the time value of money. The value of all expenditures and revenues in the past and in the future is determined at one definite time, May 1st 1996. For a series of annual payments the net present value is given in equation 44 (Brouwer, 1993; Blokland and Trifunovic, 1994).

$$W_h = A_{pay} * \frac{1-(1+r)^n}{r} \quad \textbf{(44)}$$

where: W_h = net present value [US$];
A_{pay} = annual payment [US$];
r = real interest rate [-];

6.2.1 Summary of Costs

The total costs of the alternatives is the sum of the investment and annual costs of the treatment system, and the investments and annual costs of the irrigation scheme. The total costs of the reuse scheme with primary effluent are lower than those of alternatives 1 and 3.

Treatment System
The investment costs of the reuse options are much lower than those of the no reuse option. This is due to the large expenditure on foreign machinery in the activated sludge plant. The annual costs of alternative 1 and alternative 3 are similar. The treatment costs of alternative 2 are negligible.

Irrigation Scheme
The investment cost of the irrigation scheme for the reuse options (1 and 2) are $1,200,000, the old irrigation scheme does not require any investments. The annual costs of all alternatives are similar. The investment costs are much lower than the annual costs.

Table 30: Costs of alternatives

		Reuse; pond system	Reuse; primary effluent	No reuse
treatment system	**investment**	$2,900,000	$35,000	$6,000,000
	annual	$3,000,000	$5,000	$2,400,000
irrigation scheme	**investment**	$1,200,000	$1,200,000	$0
	annual	$7,400,000	$7,000,000	$6,500,000
TOTAL		$13,500,000	$8,240,000	$14,900,000

6.2.2 Summary of Benefits

Treatment System
The yield of duckweed results in a small, but appreciable benefit. The direct benefits of the duckweed pond system are $600,000 in 20 years. These benefits are 10% of the costs of the irrigation scheme ($6,000,000; see table 32).

Irrigation Scheme
The benefit of the irrigation scheme is the yield of sugarcane of the irrigated area. The present yield of US$2,200,000 per year over a 20 years period, has a net present value of US$33 million. It is presumed that the price of sugarcane develops according to general inflation rates. Benefits of the reuse alternatives are 20% higher, and are US$40 million.

Table 31: Benefits of alternatives

	Reuse; pond system	Reuse; primary effluent	No reuse
treatment system	$600,000	$0	$0
irrigation scheme	$40,000,000	$40,000,000	$33,000,000
TOTAL	$41,000,000	$40,000,000	$33,000,000

6.2.3 Benefit-Cost Ratio

The reuse alternative with primary effluent is the most inexpensive; reuse with pond effluent is second best and no reuse is third. Alternative 2 is the cheapest because no expenditures are made for the treatment of wastewater. The difference between alternative 1 and 3 are mainly caused by (a) high investment costs for the activated sludge plant of alternative 3, and (b) lower benefits from the irrigation scheme of alternative 3.

Table 32: Financial Cost-Benefit Analysis

		Reuse; pond system	Reuse; primary effluent	No reuse
COSTS	treatment	$5,900,000	$40,000	$8,400,000
	irrigation	$7,600,000	$8,200,000	$6,500,000
BENEFITS	treatment	$600,000	$0	$0
	irrigation	$40,000,000	$40,000,000	$33,000,000
BENEFITS MINUS COSTS		$27,500,000	$31,500,000	$18,000,000
BENEFIT -COST-RATIO		3.04	4.88	2.21

6.3 Qualitative Checklist

Five categories of criteria are distinguished: effectivity; technical feasibility; institutional and regulatory feasibility; social preferability; and environmental preferability. These categories are discussed in the following paragraphs in the form of a qualitative checklist. The checklist consists of a list of impacts on one axis and qualitative conclusions on the other axis.

6.3.1 Effectivity
Effectivity refers to the extend that an alternative solves the problems it is supposed to solve. The effectivity of the alternatives is judged using the following question: Do contamination and depletion of natural water sources diminish after implementation of the project?

Contamination
Alternative 1 (the reuse of pond effluent) is effective: it diminishes the pollution in the river Cauca. Reuse is an alternative wastewater treatment. The effect is small, because only 4% of the wastewater of Cali is reused.

Alternative 2 does not solve contamination problems, because primary effluent is drained to the Rio Fraile. When farmers do not demand the full water requirement (and full demand only occurs every few years) the primary effluent is disposed of at the tail end of the irrigation canal into the Rio Fraile. The Rio Fraile is a small stream which is severely contaminated with a large flow of primary effluent.

The activated sludge plant in alternative 3 partly solves the contamination problems of the River Cauca. The secondary effluent of the plant contaminates the river, but to a much lower degree than the raw wastewater which is disposed at present. Complete treatment is required to solve the contamination problem in the River Cauca.

Depletion

As there is not yet a large depletion problem in the study area, none of the solutions is very effective. In the future reuse could solve growing depletion problems.

The water balances presented in appendix III show that groundwater resources are temporarily overexploited in dry periods, but that there is no permanent overexploitation. The use of surface water is established by law. Surface water is a scarce resources, which is overexploited in dry periods. The groundwater level has been measured since 1969 in the oldest well in the study area, the VP104 in Caucaseca. When analyzing the data from 1969 to 1989 there seems to be a certain drop of groundwater table, but it is not a very spectacular trend (see figure 22). Other wells show the same tendency.

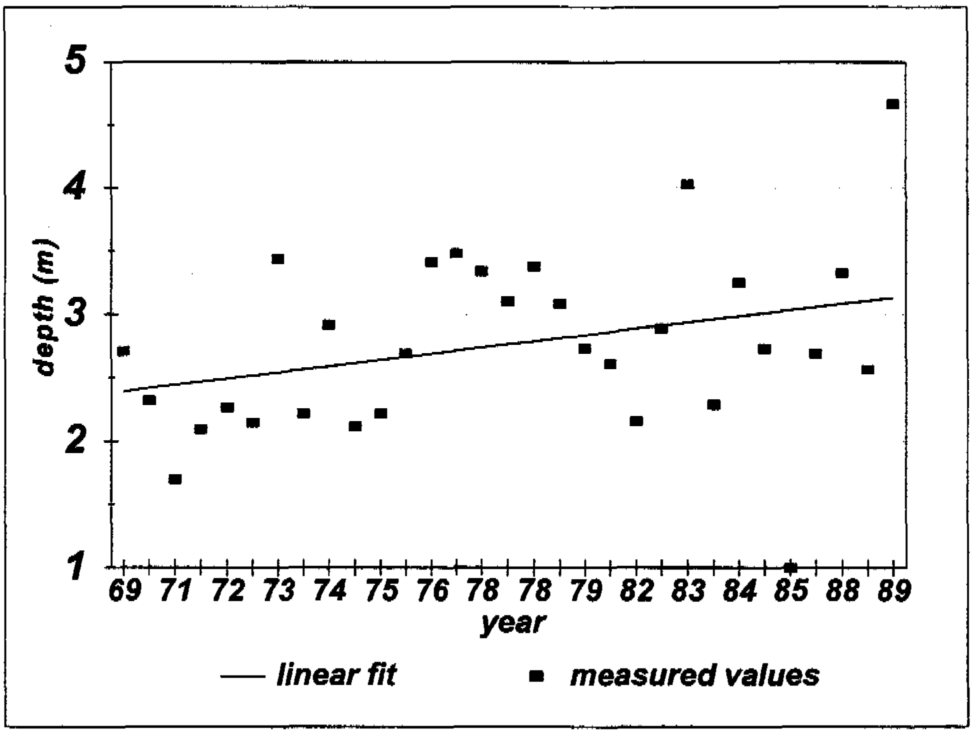


Figure 22: groundwater levels in well VP104

Data on water depletion in the study area are hard to interpret because precipitation (and therefore both use of resources and natural replenishment) varies greatly over the year. Farmers in the study area use mainly surface water for irrigation. Eight groundwater wells exploit the most superficial aquifer. Regulators and users disagree on the level of depletion of natural resources: disputes between the various users of surface water are on the increase; users of surface water say the discharges of the rivers in the Valle de Cauca have dropped over the last 30 years; the official opinion of the CVC is that discharges have not decreased, but only demand has increased. Reality is most probably somewhere in between these extremes.

Table 33: Effectivity checklist

	1	2	3
Does contamination of natural water sources diminish after implementation of the project?	+	-	±
Does depletion of natural water sources diminish after implementation of the project?	0	0	0

6.3.2 Technical Feasibility

Technical feasibility refers to the availability of resources that are necessary for the implementation of the project, like materials, man power, and skills. The theoretical basis for the treatment system, the integration of wastewater considerations in the irrigation scheme, and the availability of labor and materials are discussed in this paragraph.

Theoretical basis of Treatment

The duckweed pond system is a technical acceptable treatment option. The theoretical basis for the design of pond systems is quite strong, and there is experience of full-scale operational systems in tropical conditions. The duckweed pond system is an upgraded waste stabilization pond system. Waste stabilization pond systems have been applied locally with satisfying results. The duckweed is a most insecure component of the system, but it only removes nutrients. The duckweed pond system, although it is a new system, does not create a public health risks. Research and demonstration projects have shown that macrophyte-based systems can provide effective wastewater treatment, but full-scale systems have not often met the expectations.

The theoretical basis for the design and implementation of an activated sludge treatment system is strong. There is a lot of experience with activated sludge plants all over the world. Experience in Colombia is limited.

Integration of wastewater in irrigation system

The integration of wastewater in the irrigation scheme does not cause any problems. The salinity of the wastewater is low, so the leaching fraction can be low. Nutrients in the wastewater decrease the need to apply fertilizers. The nitrogen concentration does not pollute the groundwater.

Availability of labor

Pond systems require low-skilled personnel for operation and maintenance, and are easy to construct. Operators have to be trained in some basic skills. The implementation of an activated sludge plant is problematic in Colombia, where skilled manpower is scarce.

The implementation of an irrigation scheme using reclaimed wastewater requires some extra skills from operators and farmers of the area. They should be taught what risk the contact with the water implies, and which measures can be taken to lower risks. Farmers have to be taught that they can save fertilizers.

The implementation of a centralized irrigation scheme, in which farmers have to share water requires a change in attitude as well as new skills of the water users. The extra technical skills required for the operation and maintenance of the system consist of the operation of adjustable structures, and the measurements of discharges. The change in attitude is discussed in the paragraph on social impacts.

Availability of materials
All materials for the construction of the pond system can be locally obtained. One of the advantages of natural treatment systems is that they do not require expensive, imported parts. The irrigation system is also designed to be implemented with local materials: structures are simple and straight-forwardly designed. Construction of an activated sludge plant involves large expenditure to import materials.

Table 34: Technical feasibility checklist

	1	2	3
Does the treatment concept have a solid theoretical basis which guarantees a secure construction and operation?	0	-	+
Can the wastewater considerations be integrated into a distribution system in a secure way?	+	0	+
Are skilled labor and manpower available for the implementation of the project?	+	+	0
Are all materials for the construction of the system present or obtainable in Cali region?	+	+	-

6.3.3 Institutional and Legal Aspects

Laws and regulations
Colombian law does not state special requirements for reuse of wastewater. However, two articles of law 1594 apply to reuse: Article 40 which presents rules for the use of water resources for agriculture and article 72, which specifies the requirements for discharge of sewerage on water bodies (Republica de Colombia, 1984). It is not clear whether reuse, the discharge of wastewater into irrigation canals, must comply with the rules set for discharges of wastewater into water bodies, which are presented in article 72. The law has not been enforced up till now.

Legislative units
The reuse project is situated in two legislative units: the municipality of Cali and the municipality of Palmira. This causes large problems for the implementation of the project. The activated sludge plant treats the wastewater in the same municipality as it derives from.

In general the institutional sector is weak and not satisfactory organized to fulfill the demands of wastewater treatment plants. The administrative decentralization process, undertaken by the central government since 1986, leaves responsibilities for sanitation at a local level, at city councils. Councils of small municipalities often do not have the know-how to carry out their sanitation tasks. Sanitation used to be the responsibility of the Ministry of Transport but is now assigned to the Ministry of Economic Development, Section of Urban Development, Housing and Potable Water. In March 1995 the National Planning Department in cooperation with the National Council of Social and Economic Policy launched the Water Plan for the period of 1995-1998. The main objective of the plan is to provide the majority of Colombians with water supply and sewerage, wastewater treatment is of minor importance in the plan (Peña, 1995).

Institutional Framework
An institutional framework has to be set up to implement the reuse project. This will not be an easy task. The cooperation between the various legislative units can be expected to be difficult. ASOCAÑA and CENICAÑA can serve as a base for the water delivery organization of the irrigation scheme.

Table 35: Institutional and legal checklist

	1	2	3
Which laws and regulations exist, that the project has to obey? Do any of them forbid or complicate the implementation of the project?	0	-	+
In which legislative units is the projects located and does the project cross legislative borders?	-	-	+
Does an institutional framework exist, which can implement the project? Or how can the institutional framework be established?	-	-	0

6.3.4 Social Aspects
The social aspects of the reuse scheme include public health risks, the acceptance of reuse in the Valle de Cauca, and the participation of farmers in the centralization of the irrigation system.

Public Health Risks
The public health risks of the use of pond effluent are low: the water quality meets the strict guidelines of the WHO. The use of primary effluent does create public health risks.

Public health risk of the reuse of wastewater in sugarcane irrigation can be controlled because (1) sugarcane is not eaten raw, (2) irrigation stops three month before harvesting, and (3) sugarcane consumes a vast part of the nitrogen in the irrigation water.

The public health risks of the use of primary effluent include:
- Exposure of workers to the helminth eggs;
- Breeding places for mosquitos in the septic water in the intermittent irrigation scheme;
- Exposure of consumers, when farmers irrigate other crops than sugarcane;

Acceptance of Reuse
Reuse is accepted if it is an economical alternative. Cultural prejudice against reuse does not exist. On the other hand people are hardly conscious of a need to achieve ecologically sound farming practices and patterns of human existence. The awareness of the undesirability of polluting rivers with untreated or inadequately treated sewage is low. Farmers are willing to look for substitutes for expensive fertilizers that are required in large volumes in sugarcane cultivation. Policymakers have prejudices regarding reuse and natural treatment systems.

In large parts of the world the transformation of values has now proceeded to the point that constraints upon effective reuse are more a question of cost and technical feasibility (particularly the problem of mixing domestic and industrial wastewater in most urban systems), than a question of cultural predisposition (Feachem et al, 1983).

Centralization of the Irrigation Scheme
The centralization of the irrigation scheme involves the education of farmers, as farmers are use to irrigation per farm. The organization of the farmers is strong, so the educational problems can be overcome.

Local Community
The benefits for the local communities are moderate for all alternatives. Alternatives 1 offers some uneducated jobs. In all alternative local personnel can be educated to fulfill more difficult jobs. Alternative 2 causes odor problems to people living nearby the irrigation scheme. Alternative 1 and 2 are self-sufficient.

Table 36: Social checklist

will the implementation of the alternative:	1	2	3
create a public health risk?	+	0[1]	+
be socially accepted in the Valle de Cauca?	0	0	+
create jobs and/or income for local communities?	+	0	0
create infrastructure or services which can be used by local communities?	0	0	0
offer education to local community members?	+	+	+
occupy or affect areas which are of major significance for human settlement, agriculture, animal husbandry or similar?	0	0	0
create nuisance conditions to people living nearby (smells, noise etc.)?	0	-	0
be self-sufficient; how is looked at ways to meet each need at a more local level?	+	+	-
be vulnerable for theft, vandalism, or terrorism?	0	0	0
be manageable for the farmers?	+	+	+

1.positive effect by 'land-treating' wastewater; negative effect by exposing workers

6.3.5 Environmental Impacts
The implementation of a reuse scheme with pond effluent (alternative 1) causes the least environmental problems and some environmental benefits. The contamination problems of alternative 2 are discussed in the paragraph $5.3.1 on effectivity. The reuse of primary effluent does not cause other major environmental problems. The environmental benefits of reuse include the provision of an alternative water and nitrogen resource, and the reduction of salinity problems by leaching land with low salinity water. The implementation of alternative 3 causes less benefits, but still is beneficial when compared to the present situation.

Table 37: Environmental checklist

will the implementation of the alternative:	1	2	3
lead to the development of an unwanted disposal site, which displaces the original wildlife in the area?	0	0	0[1]
occupy or affect areas which support animal or plant life worthy of protection or especially vulnerable ecosystems?	0	0	0
flood or affect areas with historic remains or landscape elements which are of importance to the population?	0	0	0
lead to pollution of air, water or soil?	+	-	+
drain rivers or change the flow of water in such a way that it creates considerable changes for the environment and the utilization of natural resources?	+	+	0
require intensive use of non-renewable energy sources?	0	0	-
lead to felling of trees for fuel etc. which is larger than the rate of growth?	0	0	0
create major demands on other forms of infrastructure?	0	0	0
cause a noticeable reduction in the flow of nutrient elements or fish production?	+	+	0
lead to substantial waterlogging or salinity of cultivated or cultivable land?	+	+	0

6.4 Discussion

Alternative 1 and 3 are feasible alternatives. This judgment is based on the combination of the financial costs and benefits and the aspects of the checklist. Alternative 1 (reuse with pond effluent) has a slightly more favorable score than alternative 3 (no reuse). Alternative 2 (reuse with primary effluent) is inexpensive but poses unacceptable public health risks. Table 38 summarizes the scores of the alternatives.

Based on the general evaluation in this report, it can only be said that alternatives 1 and 3 are competitive. The choice between alternative 1 and 3 is a political choice, for which further investigation is required. The costs-benefit analysis of §6.2 contains very rough estimates. The checklist in §6.3 only gives a qualitative judgment of the alternatives.

Alternatives

Alternative 1 is feasible. It involves fairly low expenditure, creates high irrigation returns, and does not cause harmful impacts. The institutional and legal problems put a serious constraint on the implementation of reuse projects.

Alternative 2 is cheap but not effective. It combines low treatment costs with high irrigation benefits, but creates institutional, social and environmental problems. The primary effluent in alternative 2 is not treated prior to irrigation. Part of the primary effluent will be disposed of at the tail end of the irrigation canal in the Rio Fraile.

Alternative 3 is feasible. It has a lower benefit-cost ratio than the other alternatives, but guaranties a higher technical feasibility and less institutional and legal problems. It does not create social and environmental costs and benefits.

Table 38: Scores of the Alternatives

	Reuse; pond system	Reuse; primary effluent	No reuse
Costs	$13,500,000	$8,200,000	$14,900,000
Benefits	$41,000,000	$40,000,000	$33,000,000
Effectivity	+	-	0
Technical Feasibility	0	0	+
Institutional and Legal Aspects	-	-	0
Social Aspects	0	0	0
Environmental Aspects	+	0	0
TOTAL	+	-	+

Uncertainties
The estimates on which the evaluation in this chapter is based are very rough: the designs are general, and prices are derived from various, sometimes outdated sources. Rest values are not taken into account. The largest uncertainties are the land price and the cost of activated sludge treatment.

The rest value of the alternatives is presumed to be zero after the 20 years life span. In reality this is not true, especially for the land on which the treatment system is built: the terrain is located near the expanding city of Cali, and is more likely to rise in price than to become worthless.

The presumption that the land price is US$10 is based on estimates of all parties involved in the project. In personal communications with *ingenios*, the CVC, EMCALI, CINARA, and CENICAÑA prices between US$2 and US$20 were mentioned. A land price of US$2 would decrease the cost of alternatives 1 and 2 by US$2,00,000, a US$20 land price would increase the cost by US$2,500,000.

The estimation of the price of the activated sludge plant is very general: it is based on a figure given without a design or prior experience. The price of the activated sludge plant is a very large factor in the comparison between the reuse and the no reuse alternatives.

7 Conclusions and Recommendations

7.1 Results

Water balances for surface water and groundwater of the Valle de Cauca are made.

A design for a reuse scheme of the effluent of the Cañaveralejo plant is made. This design includes a duckweed pond system for a flow of 475 l/s, which is 4% of the wastewater of Cali. An alternative irrigation scheme which uses the duckweed pond effluent is made with a size of 936 ha, which is 0.5% of the agricultural area of the Valle de Cauca.

An evaluation is made of the three alternatives:

1. Reuse with pond system;
2. Reuse with primary effluent;
3. Nor reuse, activated sludge treatment, present unimproved irrigation scheme;

The evaluation consists of a cost-benefit analysis, and a qualitative checklist, which judges effectivity; technical feasibility; institutional and regulatory feasibility; social preferability; and environmental preferability.

7.2 Conclusions

The contamination of the River Cauca creates serious public health and environmental risks. The contamination is largely caused by the disposal of raw wastewater of Cali into the River Cauca. Depletion problems do not yet occur at present in the Cali region.

The need for reuse of wastewater in the Valle de Cauca will increase in the coming years. Contamination will increase due to population growth and the rise of the standard of living. Depletion of groundwater and surface water will occur in certain areas at certain times, as water demand for agriculture, human use and industry increases. Reuse of domestic wastewater is a moderate factor in solving depletion problems: 6% of the water use in the Valle de Cauca is used for domestic uses.

The primary effluent of Cañaveralejo plant can serve as a reliable water source for irrigation after treatment with a duckweed pond system. Reuse does not put at risk public health, crops or soils.

Reuse of part of the Cañaveralejo effluent for the irrigation of sugarcane of the area between the Rio Cauca and Rio Fraile is an economically feasible and sustainable alternative to decrease effluent disposal problems of Cali. The Cañaveralejo reuse scheme is cheaper to construct, operate and maintain than the combination of an activated sludge treatment and the old irrigation scheme. The major difficulty for implementation of the project is the institutional organization. Reuse projects involve the cooperation of several organizations with split interests. The project crosses municipal borders.

Duckweed pond systems are technical feasible for large flows of domestic wastewater. The use of subsurface flow wetlands for the treatment of large urban wastewater flow is not yet a feasible option, as knowledge and experience are limited. Especially experience under tropical conditions with functioning, design, operation and maintenance of subsurface flow wetlands is very scarce. The subsurface flow wetland concept is promising for small treatment systems in Colombia, where temperature is constant and high year-round.

The effluent of a duckweed pond system influences sugarcane irrigation by providing nutrients. The wastewater provides part of the nutrient demand of the sugarcane, so less fertilizer has to be applied; but maintenance of canals and structures increases due to weeds.

Reuse projects are more attractive in areas where presently groundwater is used for irrigation. In these area the depletion of natural water sources is severe and pumping costs are higher.

An improved centralized irrigation scheme is more economical than unimproved individual farm irrigation schemes, with or without wastewater reuse. An improved furrow irrigation system with flexitubes as tertiary canals, larger furrow spacing, and lined canals can halve water demand.

7.3 Recommendations

The disposal of the raw wastewater into the River Cauca should be put to an end as soon as possible. Every effort should be taken to put the various wastewater treatment plants in the Valle de Cauca into operation shortly. Further measures to reduce contamination should be planned and implemented. The CVC should enforce the regulations on disposal of wastewaters.

Measurements should be better documented by the various responsible organizations. A centralized database system should be established, in which all data on quantitative and qualitative water management are collected.

Pilot plants for macrophyte-based treatment systems should be established. These systems should also be implemented in small communities. Performance should be monitored and documented.

Reuse possibilities in other areas in the Valle de Cauca, where more groundwater is used should be investigated. An example of an area that suffers from depletion is the community of Candelaria.

A water and chloride balance should be made for the Valle de Cauca. These balances could be the start of an integral water management system for the entire basin.

ASOCAÑA/CENICAÑA should investigate possibilities for centralized irrigation schemes. In this study a centralized irrigation scheme turned out to be more economical, regardless of the water source.

More knowledge about the influent and the effluent of future wastewater treatment plants ease further planning of treatment or reuse options. EMCALI should execute a research project into the quality of raw wastewater. Measurements should include discharges, BOD, TSS, nutrients, electric conductivity, SAR, chloride, and heavy metals.

All parties, involved in reuse in the Valle de Cauca, should establish an organization to investigate and implement reuse possibilities. These parties include EMCALI, ACUAVALLE, CVC, municipalities, and CENICAÑA/ASOCAÑA. These organizations could be supported by research institutes, like the University del Valle, CINARA, and CIPAV.

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APPENDICES

Appendix I: List of Symbols

α	fractional augmentation	-
ΔH	decrement	m
ΔQ	flow gain	m^3/d
ΔS_{sw}	Storage of salts	g
η	efficiency	-
λ_s	areal loading rate	g/m^2*d
λ_v	volumetric loading rate	g/m^3*d
μ	contraction coefficient	-
ρ	density	kg/m^3
τ_w	kinematic viscosity of water	m^2/s
A	surface area	m^2
A_{ap}	surface area of anaerobic pond	m^2
A_c	cross-sectional area of wetland	m^2
Adj R_{Na}	adjusted sodium adsorption ratio	-
A_{fp}	surface area of facultative pond	m^2
A_{pay}	annual payment	US$
A_s	surface area of wetland	m^2
b	bed width	m
b_{nd}	width of distributor	m
BOD	Biochemical Oxygen Demand	mg BOD/l
b_w	width of weir	m
c	weir coefficient for free flow	$m^{1/2}/s$
Ca_x	modified calcium value	me/l
C_{dw}	salt concentration of drainage water	g/m^3
C_{dw}	nitrogen concentration in drainage water	mg/l
C_e	effluent pollutant concentration	mg/l
C_{gw}	salt concentration of groundwater	g/m^3
C_{iw}	salt concentration of rain water	g/m^3
C_{iw}	nitrogen concentration of irrigation water	mg/l
C_o	influent pollutant concentration	mg/l
COD	Chemical Oxygen Demand	mg O_2/l
C_{rw}	salt concentration of irrigation water	g/m^3
D	diameter	m
D_{dw}	flow of drainage water	m^3
D_{gw}	flow of groundwater	m^3
D_{iw}	flow of irrigation water	m^3
D_m	diameter of void spaces in media	m
D_r	flow of rainwater	m^3
E	evapotranspiration	m^3/year
E_0	evaporation	mm/month
e_a	field water application efficiency	-
e_c	conveyance efficiency	-
EC	electrical conductivity	dS/m = mmho/cm
EC_{dw}	electric conductivity of drainage water	dS/m
Ec_e	tolerated electric conductivity of crop	dS/m
EC_{iw}	electric conductivity of irrigation water	dS/m
Ec_s	electric conductivity of the root zone	dS/m
e_d	distribution efficiency	-
e_p	overall efficiency	-
E_R	removal efficiency	%

f	fraction of applied nitrogen removal	-
g	gravity	9.81 m^2/s
H	head	m
HD	human discharge	m^3/year
H_f	friction head loss	m
H_l	local head loss	m
HLR	hydraulic loading rate	m/day
H_t	total head loss	m
HU_{gw}	human use of groundwater	m^3/year
HU_{sw}	human use of surface water	m^3/year
k	local head loss coefficient	-
K_{20}	first-order reaction rate constant at 20°C	d^{-1}
k_c	crop coefficient	-
K_{NH}	nitrification rate constant at 20°C	d^{-1}
k_s	Strickler roughness coefficient	$m^{1/3}/s$
K_T	temperature-dependent first-order reaction rate constant	d^{-1}
L	length	m
LF	leaching fraction	-
LR	leaching requirement	mm/month
Mg	magnesium concentration	me/l
n	side slope	-
Na	sodium concentration	me/l
n_p	porosity	-
N_R	Reynolds number	-
NR	natural recharge of groundwater	m^3/year
P	precipitation	m^3/year
$P_{0.8}$	80% dependapble percipitation	mm/month
P_{can}	wetted perimeter	m
P_{eff}	effective percipitation	mm/month
P_{ower}	power	J
Q	discharge	m^3/s
Q_0	influent flow	m^3/d
Q_{con}	discharge with continuous delivery	l/s
Q_e	effluent flow	m^3/d
Q_{in}	water entering form other aquifers	m^3/year
Q_{int}	discharge with intermittent flow	l/s
Q_{max}	design discharge	m^3/s
Q_{min}	minimum discharge	m^3/s
Q_{out}	water exiting to other aquifers	m^3/year
Q_{tr}	discharge of treatment system	l/s
r	real interest rate	-
R	hydraulic radius	m
rz	percent of bed depth occupied by root zone	-
s	hydraulic gradient	m/m
SAR	Sodium Adsorption Rate	-
S_c	amount of salt removed through crops	g
S_f	amount of salt added by agricultural chemicals	g
SF_{in}	surface flow in	m^3/year
SF_{out}	surface water flow out	m^3/year
S_{gw}	storage of groundwater	m^3/year
S_m	amount of salt dissolved from minerals in the soils	g
S_p	amount of salt precipating into the soil after irrigation	g
S_{sw}	storage of surface water	m^3/year

t	hydraulic residence time	d
T	temperature	°C
t_a	actual residence time	d
TDS	Total Dissolved Solids	mg/l
TN	Total Nitrogen	mg/l
TP	Total Phosphorus	mg/l
TSS	Total Suspended Solids	mg/l
U	nitrogen uptake by crop	g/m^2 * year
v	velocity	m/s
V_{ap}	volume of anaerobic pond	m^3
V_f	field water requirement	mm/year
V_n	maximum irrigated volume for nitrogen control	m/year
V_r	volume of reservoir	m^3
V_t	volume required for preplant irrigation	mm/month
W	width of wetland cell	m
W_h	net present value	US$
y	depth	m
Y_{old}	yield with unimproved irrigation scheme	US$/ha

Appendix II: Organizations and Addresses

ACUAVALLE	Regional Public Works Department, responsible for wastewater treatment in the urban areas in the Valle de Cauca (except Cali)
ASOCAÑA	Columbian National Association of Sugarcane Producers
CENICAÑA	Columbian National Sugarcane Research Institute
CINARA	Interregional Research Institute for Sanitary Engineering, attached to the Universidad del Valle
CVC	Regional Natural Resources Body
IGAC	National Topographic Institute
INGENIO	Sugarcane factory, which also cultivates sugarcane
EMCALI	Public Works Department of Cali, responsible for wastewater treatment
NITOGOI	Consortium of Engineering Consultancy, which made the design of the Cañaveralejo plant
UNIVERSIDAD DEL VALLE	State University of the Valle de Cauca Province

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Appendix III: Water balance of the Valle de Cauca

The general form of a water balance is "IN = OUT + STORAGE". By estimating all inward and outward factors of a water balance, the storage can be calculated. Two water balances are made: a surface water balance and a groundwater balance for the most superficial aquifer.

Surface Water balance of the Valle de Cauca

The inflows in the Valle de Cauca are precipitation, surface flow, and human discharges. The outward flow consists of evapotranspiration, surface flow, human use, and the natural recharge of groundwater. The storage should equal zero for an average year. The surface water balance can be written as:

$$P + SF_{in} + HD = E + SF_{out} + HU_{sw} + NR \pm S_{sw}$$

where:
- P = Precipitation [m³/year];
- Sf_{in} = Surface flow in [m³/year];
- HD = Human discharge [m³/year];
- E = Evapotranspiration [m³/year];
- Sf_{out} = Surface water flow out [m³/year];
- HU_{sw} = Human use of surface water [m³/year];
- NR = Natural recharge of groundwater [m³/year];
- S_{sw} = Storage of surface water [m³/year];

Precipitation (P) varies greatly with altitude, between 1000 and 4000 mm per year. By multiplying precipitation data of the various altitudes with the surface areas the annual volume of rainfall can be calculated. In the low value average rainfall is 1200mm, on the medium slopes of the mountains it is 1800 mm, the higher slopes 2500mm and at the tops 3000 mm.
$P = \sum(P_{annual} * A) = 1.2 \times 4.1 * 10^9 + 1.8 \times 7.1 * 10^9 + 2.5 \times 2.3 * 10^9 + 3.0 \times 1.1 * 10^9 = 26.8 * 10^9 m^3$

The inward surface flow (SF_{in}) only consists of the discharge of the Cauca river where it enters the geographical Valle de Cauca at Balsa, because no water enters through the orders borders of the geographical Valle de Cauca, which is the basin of the Cauca rivers. Average discharge is 193 m³/s, so $SF_{in} = 6.1 * 10^9$ m³.

Human discharge (HD) is the domestic, industrial and agricultural wastewater drained to surface water. The domestic discharges can be estimated from the installed and planned wastewater treatment capacity: in Cali this is nearly 10 m³/s, this accounts for more than half of the population in the Valle de Cauca, estimate for the complete population is 18 m³/s. Industrial are partly taken into account in the domestic resources, but there are industries with there own water supply and discharge. Industrial water is in the Valle de Cauca is 12 m³/s, of which probably 30% will be discharged into municipal sewer systems. It is assumed here that all irrigation water either evapotranspirates or percolates to the groundwater. The human discharge is the discharge available for reuse.
$HD = 5.68 * 10^8 + 3.78 * 10^8 \times 0.7 = 0.83 * 10^9 m^3$

Evapotranspiration (E) can be calculated using evaporation (1300 mm/year) and crop factors for various land uses. Meadows (k_c=0.8), woods (k_c = 0.86), cafe cultivation (k_c = 0.80), sugarcane cultivation (k_c = 0.8), other crops (k_c=0.8), urban development (k_c =1) and open water surface make up most of the surface area of the Valle de Cauca. Evapotranspiration is calculated by adding up the multiplications of crop factors, surface areas and evaporation:

$$\begin{aligned} E &= \sum k_c*A_c)*E_{pan} \\ &= (0.8 * A_{meadows} + 0.86 * A_{woods} + 0.80 * A_{sugarcane} + 0.8 * A_{other\ crops} + 1.0 * A_{urban} + A_{water}) * 1{,}300 \\ &= (0.8 * 1.56 + 0.86 * 7.19 + 0.80 * 1.63 + 0.8 * 4.10 + 1.0 * 0.20 + 0.03) * 10^9 * 1.300 \\ &= 12.25 * 10^9 * 1.300 = 15.9 * 10^9 m^3. \end{aligned}$$

The outward surface flow (SF_{out}) is the discharge of the Cauca river where it exits the geographical Valle de Cauca at Anacaro. Average discharge is 399 m^3/s, so $SF_{out} = 12.6 * 10^9 m^3$.

Human use (HU_{sw}) is the water retrieved from surface waters which is the total water use minus groundwater use. Total water use in the Valle de Cauca is approximately 220 m^3/s (Alzate, 1994). The CVC estimated the groundwater use in 1994 at 127 m^3/s.
$HU_{in} = 6.94 * 10^9 - 4.00 * 10^9 = 2.94 * 10^9 m^3$

The natural recharge (NR) of the groundwater is the sum of the infiltration of surface water into groundwater and the groundwater drained to surface water bodies. Infiltration consists of the percolation of rainwater and irrigation water and the infiltration from the water supply during supply, use and sewage. Torres (1995) estimates the total natural recharge of aquifers $3400*10^6 m^3/year$.

Although storage (S_{sw}) during the year can occur, annual surface water storage should equal zero. The surface water storage is the final value of the surface water balance, and can be calculated by adding and subtracting the other values to be $1.1 * 10^9 m^3$. This figure, which is the result of the many assumptions made in calculating the surface water balance, is 5% of the total on both sides on the balance. The water balance is in equilibrium, depletion or storage do not occur.

$$P + SF_{in} + HD = E + SF_{out} + HU_{sw} + NR \pm S_{sw}$$

$$26.8 + 6.1 + 0.8 = 15.9 + 12.6 + 2.9 + 3.4 - 1.1$$

Groundwater Balance Valle de Cauca
The subsoil of the Valle de Cauca consists of two aquifers divided by an impermeable formation (aquifuge) of mainly clays (See figure 23). Nearly all wells which are used in, exploit aquifer A, only in recent years some well are constructed to exploit aquifer C.

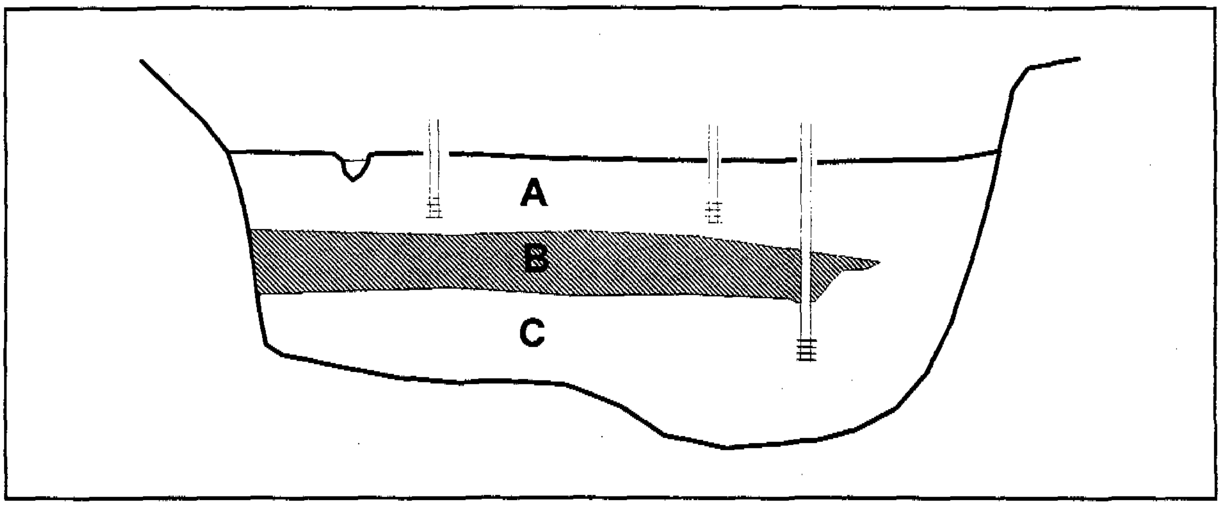


Figure 23: scheme of the subsoil in the Valle de Cauca

For aquifer A a water balance can be set up for an average year, based on the principle that flows in equals flows out plus storage. The inward flows are the natural recharge of groundwater and water entering from other aquifers, the outward flow consist of human use and water exiting to other aquifers. The groundwater balance can thus be written as:

$NR + Q_{in} = Q_{out} + HU_{gw} \pm S_{gw}$

where: NR = Natural recharge of groundwater [m^3/year];
Q_{in} = water entering form other aquifers [m^3/year];
Q_{out} = water exiting to other aquifers [m^3/year];
HU_{gw} = human use of groundwater [m^3/year];
S_{gw} = storage of groundwater [m^3/year];

The natural recharge (NR) of the groundwater is the sum of the infiltration of surface water into groundwater and the groundwater drained to surface water bodies. This is approximately $3.4 * 10^9$ $m^{3,}$ which is estimated for the surface water balance.

The water entering from other aquifers (Q_{in}) is negligible on an annual base (Martinez 1989).

The water exiting to other aquifers (Q_{in}) is negligible on an annual base (Martinez 1989).

The human use of groundwater (HU_{gw}) is the amount of water pumped from wells. The CVC estimated a well capacity of 126,886 l/s in December 1994, so HU_{gw} is $4.00 * 10^9$ m^3 per year.

In a system that is in balance the groundwater storage (S_{gw}) should equal zero. The storage in this groundwater balance equals $0.4 * 10^9$ m^3 per year, which is 10% of the total at both sides. This can either be a calculation error or a real negative groundwater storage factor, which indicates a slow depletion of the groundwater resource.

$NR + Q_{in} = Q_{out} + HU_{gw} \pm S$

$3.4 * 10^9 = 4.0 * 10^9 + 0.4 * 10^9$

There is a discussion whether aquifer A is in balance or overexploited. In dry years groundwater tables drop considerably, but in wet years most of this drop is compensated for. Data on groundwater tables are only available from commercial wells, and records are especially short in water-plentiful lower parts of the Valle de Cauca. Based on the fairly short data series one can come to the following tentative conclusions:

- groundwater levels show a slight drop in the Valle de Cauca over the past 30 years;
- during dry years aquifer A are overexploited but most of the groundwater table drop is compensated for in the wet years;
- the area of mayor concern is the area around Candelaria, where near all irrigation water comes from wells. In this area there is a serious drop in groundwater tables.

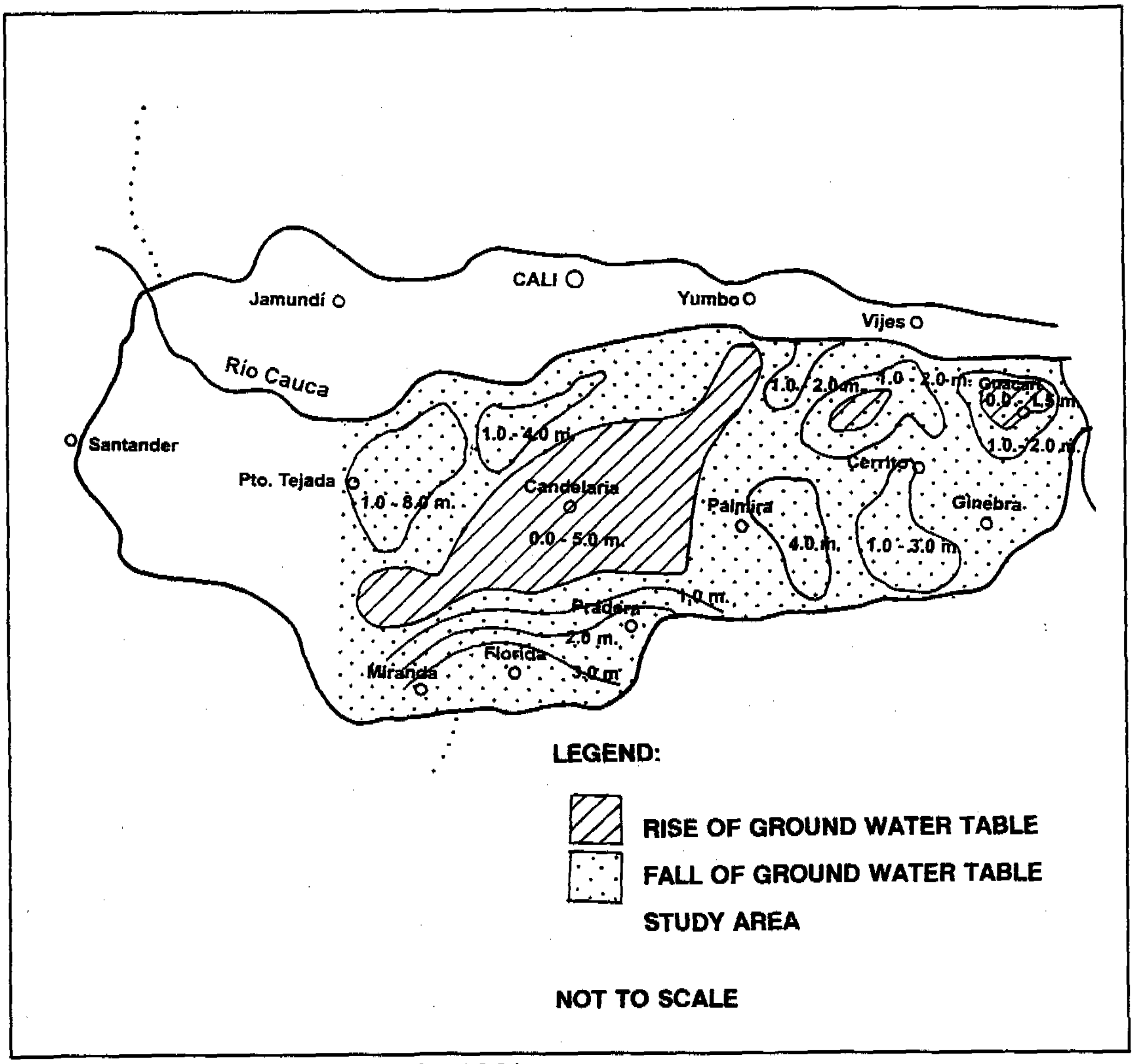


Figure 24: map of groundwater tables 87-94

Appendix IV: Climate

In terms of the Köppen's classification, the dominant climate is an Aaf-climate: A tropical forest climate with all months average above 18°C, warmest month above 22°C and rainfall in every month of the year. Rumney classifies the climate as a tropical thorn scrub woodland climate. The Valle de Cauca is located near the equator, and temperature does not fluctuate a lot during the year. There is a difference between dry and wet months in relative humidity and precipitation; the wet months (in which 60% of the rain occurs) are April/May/June, and October/November; dry months are January/February and July/August.

Climate contrasts within short distances are nowhere more widely developed than in the northern portion of tropical America, and the deep valleys and high cordilleras of Andean Colombia have a complex climatic pattern, with various micro-climates close to each other (Rumney, 1967). The weather in the Valle de Cauca varies widely from year to year. The 'Niño' phenomenon causes periods of drought. The variation in rainfall complicates irrigation planning.

Average temperatures vary from 24°C to 25°C year-round. Average relative humidity ranges from 71 to 75% for the dry and 77 to 79% for the wet months. Average sun hours vary between 160 and 175 hours per month in dry months and 145 and 160 hours in wet months. Wind velocity does not exceed 2 m/s.

Average precipitation is measured in several stations in and around the study area, The five nearest station show rainfall in the range of 850 mm to 1100 mm (see table 39 and figure 25). The Planta Rio Cauca is used in the irrigation design as it is a long series with few data missing (see table 40).

Open pan evaporation is measured in four stations near the study area and varies between 1435 and 1627 mm (see table 39 and figure 25). The series of measurements of the Granja ICA is taken as representative of the study area, as it is the longest series (see table 41).

Table 39: Availability of climatological data

station	length of series	average precipitation	average evaporation
	years	mm/year	mm/year
Guachazanbolo	22	1048	-
El Llanito	18[1]	924	-
Yunde Carrizal	29	1103	-
Matapalo	23	855	-
Planta Rio Cauca	35	1033	-
Univalle	27	-	1583
Aeropuerto	20	-	1435
CENICAÑA	13	-	1623
Granja ICA	35	-	1627

1.several years are incomplete

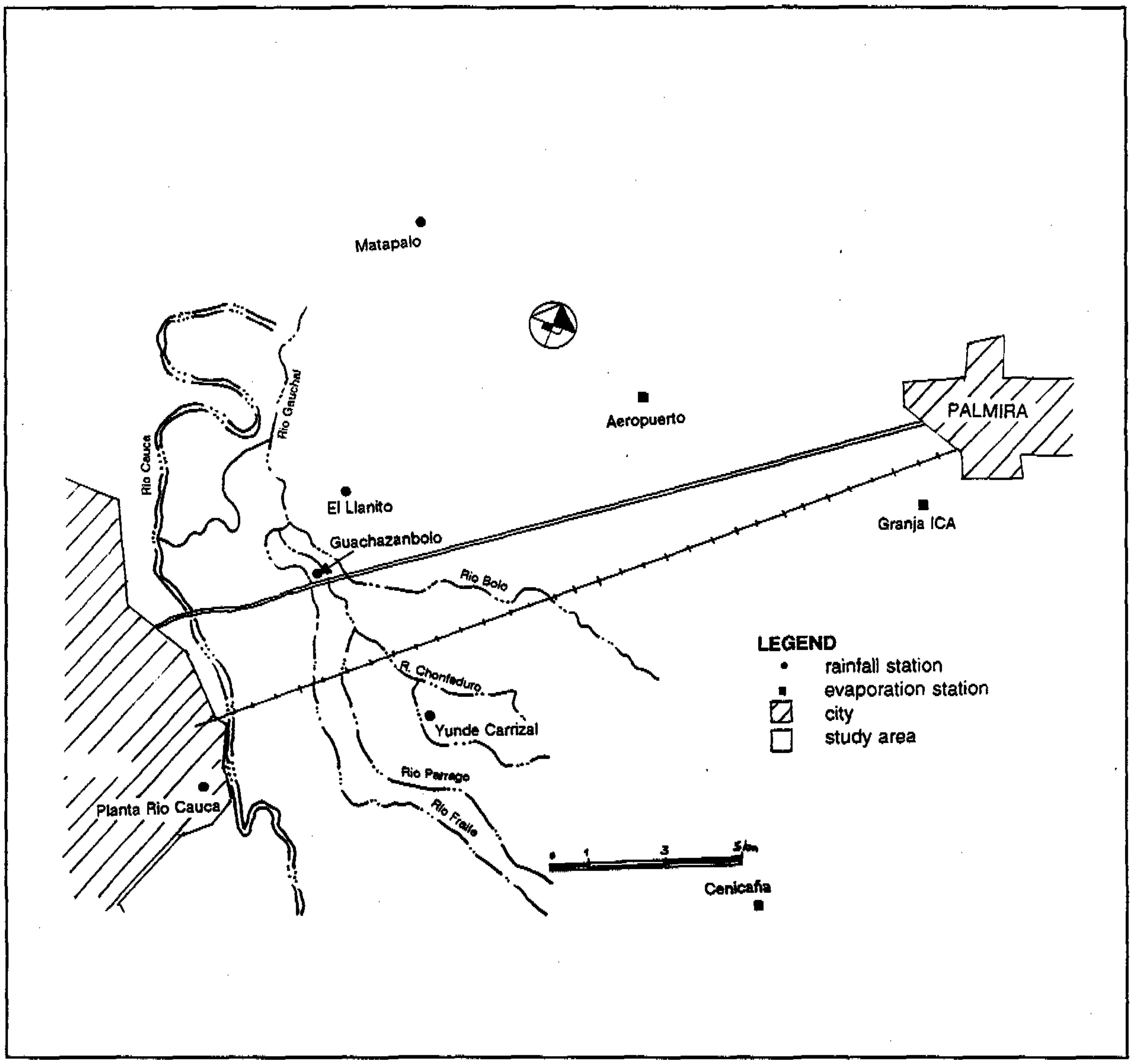


Figure 25: Climatological stations

Table 40: Precipitation Rio Cauca Station

station: Planta RioCauca						longitude	03 27 N					
municipality:	Cali					latitude 76 30 W						
						altitude 956 m						
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	year
1960	NA	NA	59*	348	143	60	146	80	56	169	111	154	1098
1961	72	NA	NA	NA	30*	80*	69*	38	36	305	151	112	714
1962	4	NA	NA	130	212	262	NA	NA	NA	NA	NA	NA	608
1963	18	216	314	214	124	[illegible]	18	NA	NA	NA	NA	NA	1904
1964	NA	NA	5	61	NA	37	20	20	42	129	44	110	468
1965	38	18	46	214	58	28	1	22	71	69	84	37	686
1966	35	55	36	42	54	66	26	54	14	133	67	74	656
1967	28	70	142	160	106	36	59	6	50	76	16*	112	845
1968	21*	84	71	154	64	113	34	56	96	176	90	48	986
1969	36	74	110	110	169	84	6	16	137	104	55	22	923
1970	44	[illegible]	20*	137	118	105	27	108	44*	82	134	45	663
1971	104	148	220	88	180	149	30	222	98	115	62	34	1450
1972	110	23	74	100	178	78	13	61	66	92	139	19	953
1973	4	35	58*	53*	46	119	67	146	[illegible]	113	65*	67	597
1974	31*	29*	43*	31	61	37	41	13	40*	113	37	28	361
1975	34	50	48	48*	50*	55	77	11*	40	111	62	61	538
1976	32	22	127	62	69*	58	4	6	26	121	125	89	672
1977	50	31	117	53	114	120	52	91	135	96	62	47	968
1978	84	53	119	360	175	19	40	126	22	66	13	80	1157
1979	49	5	199	170	6	36	53	91	52	162	64	69	956
1980	16	84	50	91	[illegible]	141	16	16	34	204	118	40*	770
1981	36	94	38*	189	174	42	28	22	53	130	142	142	1052
1982	84	130	105	202	101	65	88	3	132	214*	[illegible]	101	[illegible]
1983	18	13	118	134	43	122*	38	5	17	93	113	131	723
1984	128	268	129	116	166	76	3	90	127	151	160	57	1471
1985	110	20	78	48	90	29	13	47	165	86	106	46	838
1986	135	78	91	157	121	167	9	72	23	286	134	13	1286
1987	13	52	111	149	128	11	67	69	71	179	104	50	1004
1988	44	69	77	187	150	88	49*	54	65	125	206	84	1149
1989	88	57	93	133	133	78	17	53	55	108	141	45	1001
1990	48	88	103	113	33	29	28	12	20	180	117	133	904
1991	5	17	124	88	139	60	65	9	120	28	91	84	830
1992	20	109	50	90	54	8	44	5	107	54	105	73	719
1993	25	98	163	162	164	3	1	27	47	66	[illegible]	42	798
1994	53	129	257	210	134	36	51	22	13	147	179	27	1258
1995	55	20	165	83	129	148	173	38	52	152	94	292	1401
max	35	268	314	360	212	262	173	222	165	305	206	292	2914
avg	51	74	115	139	115	74	41	52	66	127	104	77	1033
min	4	5	5	31	6	3	1	3	13	28	13	13	125
0.7	29	34	77	90	90	37	18	18	40	94	79	46	652
0.8	18	22	63	84	58	32	13	12	26	83	62	39	512
0.9	13	18	48	54	46	21	4	6	20	66	54	27	377
#	32	30	29	32	31	33	33	33	31	32	30	33	36

Table 41: Evaporation Granja ICA

station: Granja exp ICA				longitude: 03 30 N								
municipality: Palmira				latitude: 76 19 W								
				altitude: 1006 m								
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**	**Ann**
1960	129.1	124,7	164,3	142,1	136,9	145,1	157,5	294,7	117,9	166,8	146,5	150,4	1876,0
1961	151.8	165,7	176,7	133,0	144,9	101,2	138,2	152,2	132,0	135,7	99,1	109,7	1640,2
1962	112.6	104,6	82,4	93,5	115,9	114,9	130,1	121,9	121,9	119,2	106,5	133,8	1357,3
1963	121.0	110,4	132,0	129,1	135,8	144,8	131,3	158,1	156,3	160,3	117,3	160,2	1656,6
1964	189.1	155,9	174,5	117,0	148,7	112,6	130,5	154,3	135,8	149,2	135,4	156,0	1759,0
1965	163.0	163,3	167,3	112,7	131,2	146,8	146,3	177,5	150,7	147,1	114,4	123,6	1743,9
1966	176.4	164,1	147,2	147,4	117,2	127,9	144,7	150,2	168,4	146,2	110,7	120,9	1721,3
1967	116.7	121,5	155,7	150,8	112,4	107,1	123,8	138,6	134,8	159,0	121,0	144,9	1586,3
1968	147.9	128,4	142,2	131,8	120,0	111,6	114,9	132,1	122,4	125,9	115,3	127,1	1519,6
1969	120.8	121,5	118,6	102,2	103,1	96,7	126,5	130,4	126,5	112,1	115,5	120,0	1393,9
1970	123.6	114,7	115,4	102,3	103,2	98,7	112,7	118,4	102,5	113,2	108,1	110,4	1323,2
1971	107.0	96,7	104,2	106,9	103,0	96,6	112,2	109,5	110,5	101,6	98,4	134,0	1280,6
1972	126.7	135,4	142,6	144,1	122,2	128,6	157,3	140,3	132,0	144,2	123,3	137,1	1633,8
1973	168.8	171,9	176,1	140,1	118,0	99,2	145,6	134,5	117,3	118,3	117,2	116,5	1623,5
1974	132.3	118,5	149,0	120,0	123,5	114,1	133,5	147,7	130,6	130,5	118,3	131,2	1549,2
1975	142.8	116,1	140,0	138,2	125,5	113,0	129,3	127,5	133,7	131,1	103,3	102,2	1502,7
1976	142.2	141,1	150,8	140,4	115,8	114,5	151,7	173,2	190,1	151,4	124,7	134,3	1730,2
1977	167.9	149,5	188,0	136,7	120,5	120,0	139,7	161,0	154,4	134,7	125,0	133,0	1730,4
1978	167.9	148,6	152,3	116,1	121,9	108,4	140,1	151,8	154,2	159,0	127,3	134,3	1681,9
1979	159.3	162,4	149,3	136,2	124,4	121,9	155,8	139,2	132,3	132,4	124,0	142,4	1679,6
1980	156.0	139,6	168,8	142,8	134,9	108,1	140,4	159,2	158,4	141,7	133,1	135,8	1718,8
1981	169.5	133,2	156,7	123,1	134,7	111,2	142,9	149,3	160,0	151,5	129,7	131,1	1692,9
1982	149.3	130,4	139,7	133,7	120,4		137,8	178,7	163,9	133,8	137,5	123,0	1548,2
1983	150.1	149,8	164,1	126,9	142,6	136,0	144,0	161,9	153,3	150,3	147,8	120,3	1747,1
1984	133.4	125,2	151,8	131,0	128,6	118,7	143,9	164,0	142,3	124,3			1863,2
1985	122.7	137,4	170,1	149,7	142,4	129,4	153,2	143,7	145,9	157,0	126,9	139,6	1718,0
1986	135.9	129,7	126,8	130,8	132,5	116,7	161,5	131,7	163,1	133,3	121,8	138,4	1622,2
1987	163.0	152,6	167,8	149,4	130,8	140,2	144,0	134,4	156,4	131,8	134,0	140,5	1744,9
1988	153.0	135,8	182,1	120,4	121,0	112,2	129,1	130,0	124,2	134,9	117,1	128,2	1588,0
1989	123.3	121,8	154,0	135,2	116,4	118,2	147,2	141,0	147,3	125,5	139,6	131,2	1600,7
1990	144.9	115,6	148,9	122,0	114,4	122,3	144,2	166,6	171,7	119,3	136,0	117,4	1478,4
1991	152.9	151,9	146,8	152,5	116,6	121,0	115,7	133,8	149,2	157,0	121,0	145,5	1511,0
1992	149.3	139,7	161,0	130,0	127,2	154,1	147,7	164,9	151,1	141,5	125,4	134,7	1577,3
1993	139.4	128,0	135,2	116,6	114,4	119,8	135,6	184,4	142,9	159,3	118,2	137,7	1492,1
1994	133.0	117,4	136,8	130,5	114,3	135,4	153,9	162,6	174,1	160,2	136,4	134,4	1556,0
1995	158.1	163,7	165,9	134,8	131,0	128,7	131,5	139,5					995,1
maxi	189.1	171,9	188,0	152,5	148,7	154,1	161,5	184,4	190,1	166,8	147,8	160,2	1759,0
avera	144.5	135,7	150,1	129,7	124,1	119,9	138,7	147,5	143,7	138,8	122,8	131,8	1627,4
mini	107.0	96,7	82,4	93,5	103,0	96,6	112,2	109,5	102,5	101,6	98,4	102,2	12806.

Appendix V: Soils and Land Capability

In the Valle de Cauca alluvial deposits are scattered in narrow, irregular strips bordering the river Cauca and other streams. Four soil types (Juanchito soils, Marruecos soils, Gudualito soils, and Corintias soils) dominate the Juanchito-Guanabanal-Palmaseca area, they can be classified in three orders, according to the USDA soil taxonomy: mollisols, vertisols, entisols (see table 42 and figure 26). Juanchito and Corintias soils are located in the floodplains, the lower areas in which the silt settles, with poor natural drainage. The ridges formed along river banks, consist of coarser textured parental material, like Marruecos and Gudualito soils (Cruickshank, 1972; Foth, 1978; CVC, 1980; Aristazabal, 1993). The area is flat, slopes are between 0 and 3%. There are no erosion problems. The depth of the groundwater table ranges from 50 cm to 200 cm.

Table 42: Soil types

	order	basic infiltration rate (mm/hour)
Juanchito	entisols	4.6
Corintias	vertisols	11.1
soil type	mollisols	1.1
Marruecos	mollisols	2

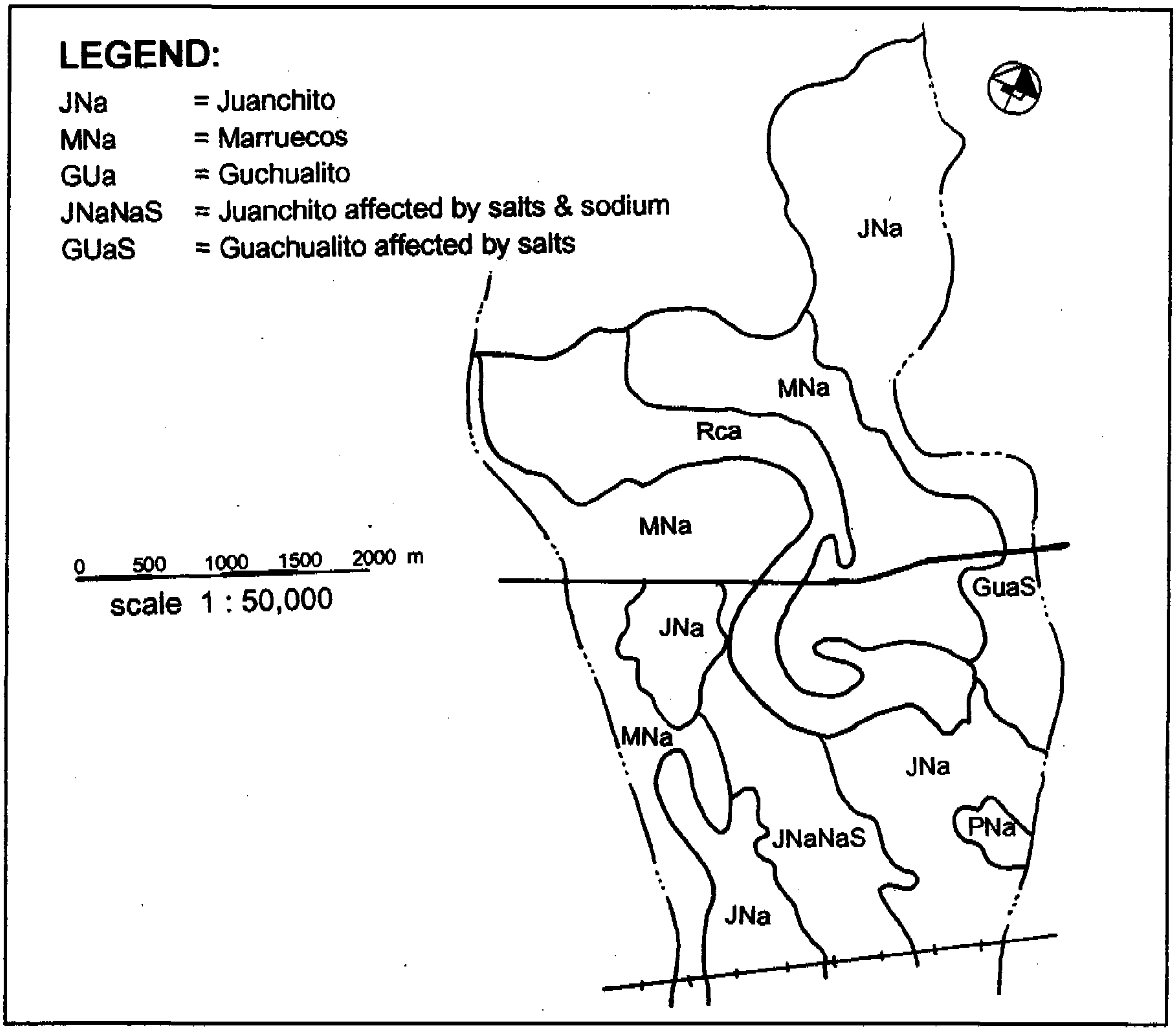


Figure 26: Soil types in Juanchito-Guanabanal-Palmaseca area

Juanchito soils are superficial to very superficial soils which can be classified as entisols. Entisols are recent mineral soils, with no distinct horizons within 1 meter of the soil surface, consisting of sediments staying behind after flooding. The Juanchito soils are found in the floodplains of the various rivers, they tend to be coarser textured (clay loam) near a stream and finer textured (loam) near the outer edges of the floodplain. Total saturation is very high, natural drainage is poor to very poor. Juanchito soils are moderately acid to lightly alkaline. Cations exchange capacity is normal to very high. SAR is high, although some areas are sodic and/or saline. Juanchito soils are used for sugarcane, sorghum, and soya cultivation and meadows.

Corintias soils are moderately deep soils of the vertisols class. Vertisols are inverted soils that are over 50 cm thick, containing over 30% of clay. Due to wetting and drying the clay cracks widely and surface particles fall into the dried-out cracks, thus inverting soil characteristics. Vertisols have a high natural fertility level and have great agricultural potential where power tools, fertilizers and irrigation are available. Corintias soils are moderately structured, consisting mainly of clay. Total saturation is very high, drainage is poor to fair. Cation exchange capacity is high to very high; carbonates are present in the upper layers. The soils are alkaline to very alkaline. Corintias soils are mainly used for sugarcane cultivation.

Marruecos are superficial to moderately deep soils, which can be classified as mollisols. Mollisols are soft soils that combine high soil fertility and fair-to-adequate rainfall. Features of mollic horizons include a thickness of 25 cm or more, dark color and at least 1% organic matter and over 50% base saturation. In the Valle de Cauca mollisols are present in dikes along the Cauca rivers and formed by recent and older sediments of this flow. Texture of the upper horizon is clay loam to sandy loam; in the rest of the profile it is clay to silty clay loam. Total saturation is high, drainage is poor to very poor. Marruecos soils are very acid to moderately acid. Cation exchange capacity is normal to very high. Marruecos soils are used for meadows, sorghum and soya cultivation.

Gudualito soils are moderately deep to deep soils of the mollisol class. Mollisols are soft soils that combine high soil fertility and fair-to-adequate rainfall. They have a well-textured upper layer of clay loam to sand. Total saturation is very high, natural drainage is fair to moderately good. Cation exchange capacity is low to very high. They are high in $CaCo_3$, and sometimes sodic. The soils are neutral to very alkaline. Land is used for cattle-breeding and sugarcane, cotton, corn and yucca cultivation.

The Valle de Cauca is a rich agricultural area. Climatological conditions are ideal for a year round use of the land, soils are fertile. In the design area sugarcane is the predominal crop grown, other crops include sorghum. meadows and annual crops (see figure 27). Due to poor irrigation practices, salinization has occurred in the drier parts of the Valle de Cauca, and some parts are not cultivated because of high salinity.

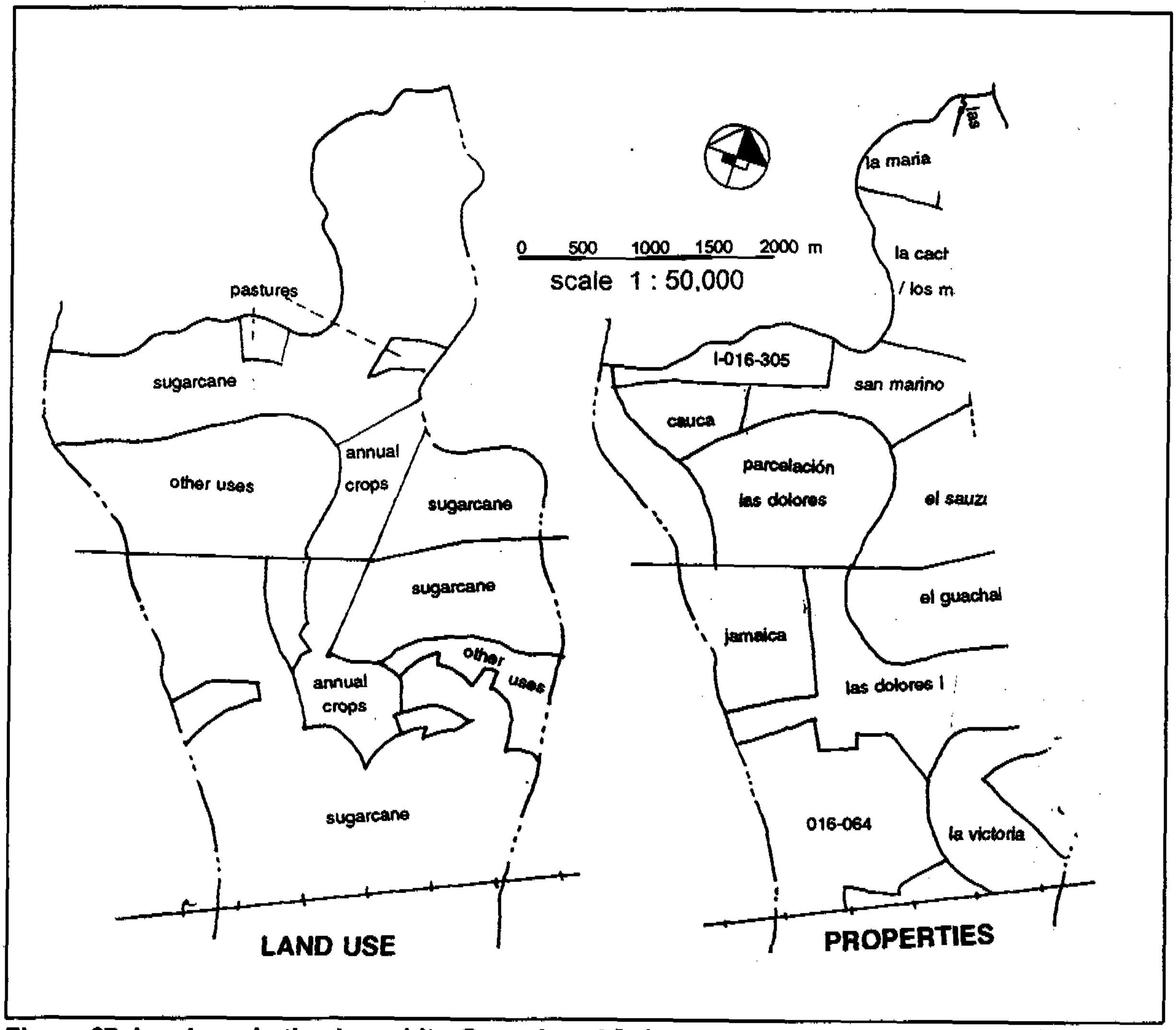


Figure 27: Land use in the Juanchito-Guanabanal-Palmaseca area

Appendix VI: Present and Planned Wastewater Treatment

The wastewater of Cali city will in the future be treated in four wastewater treatment plants: the "Planta UASB Vivero Municipal", the "Planta Cañaveralejo", the "Planta UASB Rio Cali, and "the "Planta UASB Sur". The wastewater of Cali is nearly completely of domestic sources, as most industry of the city is located in Yumbo an industrial zone which is a separate community. Yumbo wastewater is not included in this study.

The Planta Cañaveralejo will collect and treat 76% of the wastewater of Cali. Planned final capacity in 2015 is 7.6 m^3/s for preliminary and primary treatment and 3.8 m^3/s for secondary treatment. The Cañaveralejo plant will be built in the Alfonso Lopez neighborhood, adjacent to the Cauca river. The surface area available for the plant is 23 ha. The treatment plant will contain a conventional aerobic activated sludge system; the reactor is an aerated tank or basin containing a suspension of the wastewater and microorganisms (see figure 31). The content of the aeration tank are mixed vigorously by aeration devices that also supply oxygen to the biological suspension. Following the aeration step, the microorganisms are separated from the liquid by sedimentation (Asano et al, 1985). The design was made in 1990 by two Japanese and two Colombian consultancy agencies, working under the name of NITOGOI. Process parameters are presented in table 43.

The total costs to built the plant are estimated to be US$ 117.5 million, of which US$75 million will be used to construct preliminary and primary treatment and US$42.5 million to build secondary treatment. The construction of the plant is financed by the Overseas Economic Cooperation Fund of Japan, which provided a loan of US$167.5 million to EMCALI, aimed for programming of improvement of water supply and sewage systems in Cali. From this loan 80% will be used for the Cañaveralejo plant. Condition of the loan is that construction is carried out before 1998. The bidding for the construction of the plant started in December 1995, construction will start in the fall of 1996.

The influent of the Cañaveralejo plant is collected in three pumping stations and a main sewer. The three pumping stations (Cañaveralejo and Navarro, which are in operation and Aguablanca which is planned) transport pure domestic wastewater. The water in the main sewer (Colector General) is mixed domestic and industrial wastewater.

Table 43: Design parameters of Cañaveralejo plant (Nitogoi, 1990)

parameter	primary influent	primary effluent
	mg/l	mg/l
BOD total	167	106
BOD dissolved	71	41
Total Suspended Solids	224	129
Votile Suspended Solids	165	93
Total Kjeldahl Nitrogen	-	26
Total Phosphorus	-	3

The Vivero plant is the only wastewater treatment plant in operation in Cali. It treats a wastewater flow of 20.000 habitants equivalents, design discharge is 45 l/s. It uses two upflow anaerobic sludge blanket tanks, with a volume of 500 m^3 each. In an UASB reactor treatment occurs as wastewater, flowing upward through a sludge blanket, comes in contact with the sludge granules.

The Rio Cali will use an upflow anaerobic sludge blanket system. The capacity of the plant will be 2.0 m^3/s when completed in 2015 and 0.9 m^3/s initially after construction, which is planned to be completed in 1999. BOD-load will be 12.83 ton BOD/day. EMCALI is at present looking into post-treatment options for the Rio Cali plant, including aerated lagooning, rotating bio disks, and filter columns.

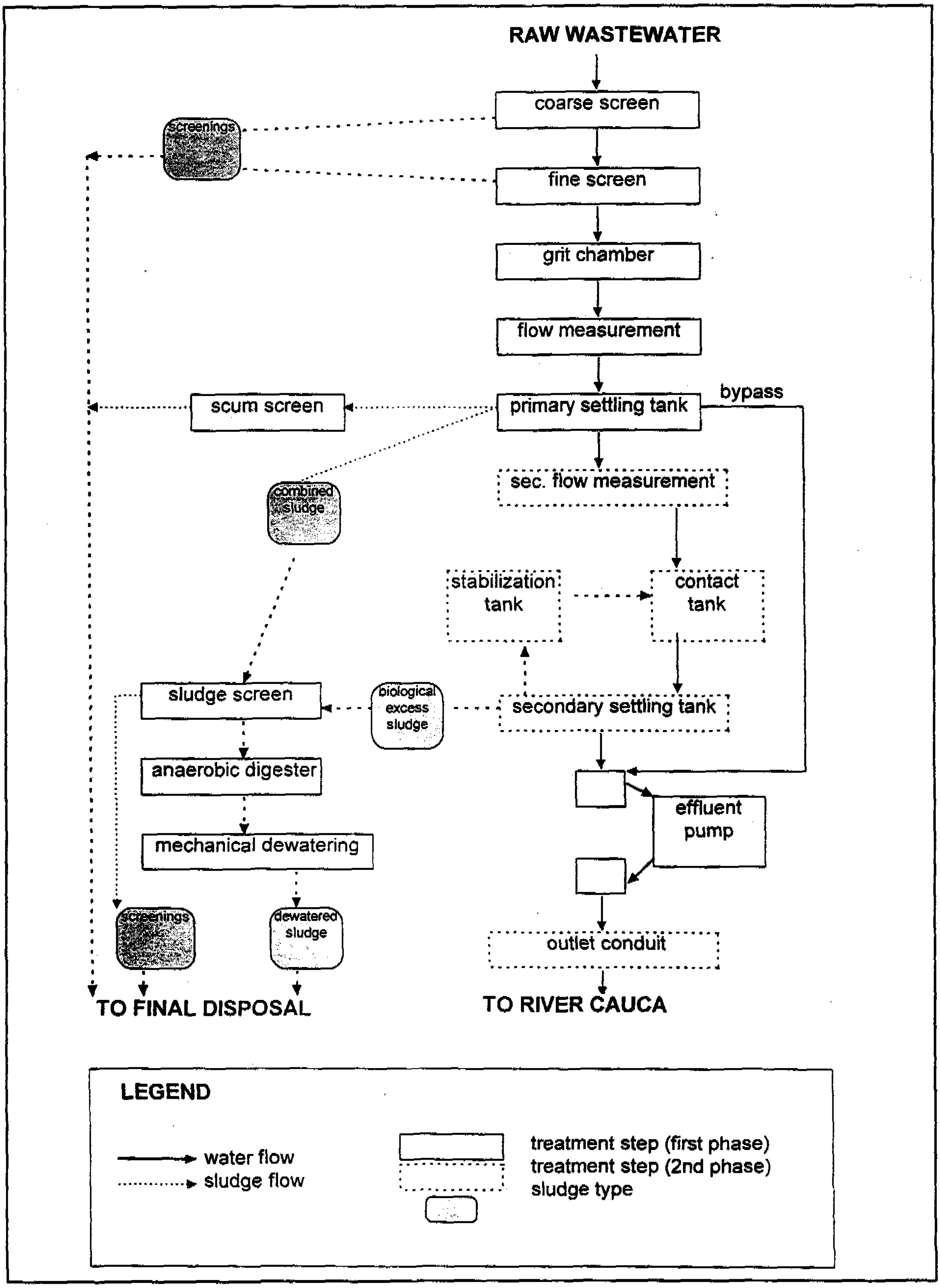


Figure 28: Plant scheme Cañaveralejo plant

The Planta UASB Sur will use an upflow anaerobic sludge blanket system as well. The planning of the plant is not yet completed and it will not be constructed in the near future.

The CVC (Corporation Autonoma Regional del Cauca) is the departmental organization which controls water contamination. The basic requirements for disposal of wastewater effluents in the Rio Cauca are set in the law 1594 (Republica de Colombia Ministerio de Salud, 1984) and Acuerdo No.14 (CVC, 1976). The requirements are a labyrinth of strict site specific regulations, regarding BOD, temperature, organic compounds, toxicity, pathogens, pH, and suspended solids. It would take a tremendous effort to enforce this system; it has not been enforced in any manner up till now. The CVC document (CVC, 1976) further states that the Water Division of the CVC can establish limits on irrigation water quality, in relation to variations in soils, cultivated crops, irrigation practices and other parameters. These limits would include SAR, electric conductivity and pathogens. Special attention would be paid to global and local economy affected by the standards. Up till now no irrigation water quality criteria other than the federal once are set. In 1976 the CVC required EMCALI to provide preliminary treatment from 1981, primary treatment from 1985, and secondary treatment of all wastewater from 1990 on. This has not been realized. Industries were also obliged to take wastewater treatment measures. This measure has not been enforced either.

Experience with Wastewater Reuse

Some experience on the reuse of wastewater exists in the Valle de Cauca. In a few small communities the effluent of waste stabilization pond is reused in irrigation. Besides there is said to be some illegal direct reuse.

There are presently five wastewater stabilization ponds in operation or under construction by ACUAVALLE. ACUAVALLE is the regional equivalent of EMCALI and takes care of the wastewater treatment in the urban zones of the Valle de Cauca, except Cali. The Ginebra wastewater stabilization pond offers effluent to nearby farmers and is a good example of the system used.

The Ginebra wastewater stabilization ponds treat the wastewater of the 11.000 people living in the community of Ginebra, a village 59 km North-West of Cali. The stabilization ponds have been in operation for 2½ years now, but measurements on influent and effluent have just started to be carried out in December 1995. Discharge at the inlet is 20 l/s. at the outlet it is 18 l/s, as 2 l/s is lost due to evaporation and infiltration. Effluent is used for sugarcane, rice, sorghum and vineyard irrigation. The wastewater stabilization ponds are not specially designed for wastewater reuse for irrigation, and their design only considers BOD load. The wastewater system consists of an anaerobic pond with a surface area of 0.138 ha and a depth of 4 m and a facultative pond with a surface area of 0.632 ha and a depth of 1.8 m. Hydraulic retention times are 2 and 6 days. BOD- removal is about 80%, influent concentration is 209 mg/l, effluent will be about 58 mg/l (Ingenieros Consultaros Ltda and Incol Ltda, 1990).

Peña (1995) cites prices of the construction of four wastewater stabilization ponds in the Valle de Cauca (see table 44).

Table 44: Costs of waste stabilization pond systems in the Valle de Cauca (Peña, 1995)

	Total costs (US$)	Unit costs (US$/m^3.day)
Guacari	241,000	43
Ginebra	467,000	270 (including pumping)
Roldanillo	268,000	56
La Union	282,000	52
Toro	200,000	90

Table 45: Measurements in the Colector General

date	BOD	TSS	TKN	TP	Cl	Zn	Ni	Cu	Cd	Pb	EC
	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	mg/L	dS/m
average	244	261	29,309	3,900	67,525	0,319	0,397	0,045	0,054	0,096	0,513
# data	28	28	8	2	8	43	10	31	10	11	6
90						0,855	0,800	0,100	0,104	0,020	
90						0,042	0,900	0,100	0,092		
90						0,676	0,700	0,050	0,079		
90						0,389		0,008	0,054		
90						1,884		0,024	0,066		
90						0,346		0,019	0,015		
90						0,190		0,024	0,015		
90						0,225		0,030	0,066		
90						0,539		0,041			
90						0,142		0,041			
90						0,202		0,041			
90						0,034					
90						0,298					
90						0,334					
90						0,502					
90						0,394					
90						0,250					
90						0,354					
90						0,106					
90						0,507					
08-91			30,6	5,5	66,500	0,100	0,040	0,018			
08-91			26,8	2,3	56,200	0,080		0,011			
16-03-93	153	864				0,200					
16-03-93	153	564				0,300					
16-03-93	302	282				0,030					
17-03-93	147	256				0,190					
17-03-93	359	310				0,210					
29-06-93	220	224									
29-06-93	269	232									
30-06-93	333	146									
30-06-93	477	218									
16-02-94	319	380									
03-07-94	168	368	31,68		77,500	0,125		0,050		0,425	
03-07-94	227	163	24,26		70,000	0,100		0,025	0,025	0,275	
04-07-94	283	230	35,77		51,000	0,250		0,025			
04-07-94	410	130	29,80		63,000	0,150		0,025		0,025	
07-07-94	183	306				0,625		0,025			
07-07-94	219	140				0,375		0,075		0,050	
08-07-94	67	217				0,250		0,025		0,025	
08-07-94	53	333				0,500		0,050		0,025	
17-07-94	155	200	23,81		78,500			0,075	0,025		
17-07-94	191	310	31,77		77,500	0,250		0,050			
04-08-94	217	188					0,025	0,025			
04-08-94	216	190				0,275	1,300	0,075		0,050	
05-08-94	297	60				0,200		0,100		0,075	
05-08-94	308	76				0,125		0,050		0,050	
09-02-95	299	288				0,380	0,080	0,060			
09-02-95	306	236				0,280	0,020	0,060		0,040	
10-02-95	262	230				0,220	0,060	0,060			
10-02-95	241	162				0,220	0,040	0,040			
20-02-96											0,52
20-02-96											0,57
01-03-96											0,43
08-03-96											0,61
12-03-96											0,42
02-04-96											53

Appendix VII: Irrigation Water Quality Guidelines

Table 46: Recommended microbiological quality guidelines for wastewater use in agriculture (WHO, 1989)*

category	reuse conditions	exposed group	intestinal nematodes[1] per liter	Faecal coliforms per 100 ml	Wastewater treatment expected to achieve microbiological quality
A	Irrigation of crops likely to be eaten uncooked, sports fields, public parks[2]	Workers, consumers, public	≤1	≤1000	A series of stabilization ponds designed to achieve the microbiological quality indicated, or equivalent treatment
B	Irrigation of cereal crops, industrial crops, fodder crops, pasture and trees[3]	Workers	≤1	Not applicable	Retention in stabilization ponds for 8-10 days or equivalent helminth and faecal coliform removal
C	Localized irrigation of crops in category B if exposure of workers and the public does not occur	None	Not applicable	Not applicable	Pretreatment as required by the irrigation technology, but no less than primary sedimentation

* In specific cases., local epidemiological, socio-cultural and environmental factors should be taken into account, and these guidelines modified accordingly.

1. *Ascaris, Trichuris* and hookworms.

2. A more stringent guideline (≤200 faecal coliforms/100 ml) is appropriate for public lawns, such as hotel lawns, with which the public may have direct contact.

3. In the case of fruit trees, irrigation should cease two weeks before fruit is picked, and no fruit should be picked off the ground. Sprinkler irrigation should not be used.

Table 47: Guidelines for interpretations of water quality for irrigation (Ayers and Westcot, 1985)*

Potential Irrigation Problem	Units	Degree of Restriction on Use		
		None	Slight to Moderate	Severe
Salinity				
EC_w or	dS/m	< 0.7	0.7 - 3.0	> 3.0
TDS	mg/l	< 450	450 - 2000	> 2000
Infiltration				
SAR = 0-3 and EC_w =		> 0.7	0.7 - 0.2	< 0.2
= 3 - 6		> 1.2	1.2 - 0.3	< 0.3
= 6 - 12		> 1.9	1.9 - 0.5	< 0.5
= 12 - 20		> 2.9	2.9 - 1.3	< 1.3
= 20 - 40		> 5.0	5.0 - 2.9	< 2.9
Specific Ion Toxicity				
Sodium (Na)				
surface irrigation	SAR	< 3	3 - 9	>9
sprinkler irrigation	me/l	< 3	> 3	
Chloride (Cl)				
surface irrigation	me/l	< 4	4 - 10	> 10
sprinkler irrigation	me/l	< 3	> 3	
Boron (B)	mg/l	< 0.7	0.7 - 3.0	> 3.0
Trace Elements				
Miscellaneous Effects				
Nitrogen (NO_3 - N)	mg/l	< 0.5	5 - 30	> 30
Bicarbonate (HCO_3)	me/l	< 1.5	1.5 - 8.5	> 8.5

Table 48: Recommended maximum concentrations of trace elements in irrigation water

Element		Maximum Concentration (mg/L)				Remarks
Code	Name	USA[1]	SA[2]	FAO[3]	COL[4]	
Al	aluminum	5.0	5.0	5.0	5.0	Can cause nonproductivity in acid soils (pH < 5.5). but more alkaline soils at pH > 5.5 will precipitate the ion and eliminate any toxicity.
As	arsenic	0.10	0.10	0.10	0.10	Toxicity to plants varies widely, ranging from 12 mg/L for Sudan grass to less than 0.05 mg/L for rice.
Be	beryllium	0.10	0.10	0.10	0.10	Toxicity to plants varies widely, ranging from 5 mg/L for kale to 0.5 mg/L for bush beans.
Cd	cadmium	0.01	0.01	0.01	0.01	Toxic to beans, beets and turnips at concentrations as low as 0.1 mg/L in nutrient solutions. Conservative limits recommended because of its potential for accumulation in plants and soils to concentrations that may be harmful to humans.
Co	cobalt	0.05	0.05	0.05	0.05	Toxic to tomato plants at 0.1 mg/L in nutrient solution. Tends to be inactivated by neutral and alkaline soils.
Cr	chromium	0.10	0.10	0.10	0.10	Not generally recognized as am essential growth element. Conservative limits recommended because of lack of knowledge on toxicity to plants.
Cu	copper	0.20	0.40	0.20	0.20	Toxic to a number of plants at 0.1 to 1.0 mg/L in nutrient solutions.
F	fluoride	1.0	2.0	1.0	1.0	Inactivated by neutral and alkaline soils.
Fe	iron	5.0	5.0	5.0	5.0	Not toxic to plants in aerated soils but can contribute to soil acidification and loss of reduced availability of essential phosphorus and molybdenum. Overhead sprinkling may result in unsightly deposits on plants, equipment and buildings.
Li	lithium	2.5	0.07	2.5	2.5	Tolerated by most crops up to 5 mg/L; mobile in soil. Toxic to citrus at low levels (< 0.075 mg/L). Act similarly to boron.
Mn	manganese	0.2	0.20	0.20	0.20	Toxic to a number of crops at a few tenth mg to a few mg/L, but usually only in acid soils.
Mo	molybdenum	0.01	0.01	0.01	0.01	Not toxic to plants at normal concentrations in soil and water. Can be toxic to livestock if forage is grown in soils with high levels of available molybdenum.
Ni	nickel	0.2	0.02	0.20	0.20	Toxic to a number of plants at 0.5 to 1.0 mg/L; reduced toxicity at neutral or alkaline pH.
Pb	lead	5.0	0.10	5.0	5.0	Can inhibit plant cell growth at very high concentrations.
Se	selenium	0.02	0.02	0.02	0.02	Toxic to plants at concentrations as low as 0.025 mg/L and toxic to livestock if forage is grown in soils with relatively high levels of added selenium. An essential element for animals but in very low concentrations.

Sn	**tin**	-	-	-	-	Effectively excluded by plants; specific tolerance unknown.
Ti	**titanium**	-	-	-	-	Effectively excluded by plants; specific tolerance unknown.
W	**tungsten**	-	-	-	-	Effectively excluded by plants; specific tolerance unknown.
V	**vanadium**	0.10	-	0.10	0.10	Toxic to many plants at relatively low concentrations.
Zn	**zinc**	2.0	4.0	2.0	2.0	Toxic to many plants at widely varying concentrations; reduced toxicity at pH > 6.0 and in fine-textured or organic soils.

1.Pescod, page 119

2.Saudi Arabia, Pescod, page 275

3.Food and Agricultural Organization of the United Nations, Ayers, p 96

4.Republica de Colombia, Ministerio de Salud 1984, Article 40 Agricultural use

Table 49: Colombian standards for wastewater discharge onto water bodies (Ministerio de Salud, 1984)

Parameter	existing users	new users
pH	5-9	5-9
temperature	≤ 40 °C	≤ 40 °C
floating materials	absent	absent
fats and oils	removal of ≥ 80%	removal of ≥ 80%
suspended solids	removal of ≥ 50%	removal of ≥ 80%
COD (domestic sources)	removal of ≥ 30%	removal of ≥ 80%
COD (industrial sources)	removal of ≥ 20%	removal of ≥ 80%

Table 50: Summary of Israeli standards for irrigation with wastewater effluents (Shelef et al, 1987)

Parameter	unit	Crops			
		A	B	C	D
Total BOD	mg/l	60	45	40	25
Dissolved BOD	mg/l	-	-	20	15
Suspended Solids	mg/l	50	40	35	20
Total Coliforms	/100 ml	-	-	100	12
Dissolved Oxygen	mg/l	0.5	0.5	0.5	0.5
Cl_2 contact minutes	min			60	120
Cl_2 residual after 60 min contact	mg/l	-	-	0.15	0.5
Minimum distance to residential area	m	300	250	-	-
Minimum distance to paved road	m	30	25	-	-

Legend to crops:

A : Cotton, sugar-beet, grains, seeds, dried fodder and other crops. Forest and wooded areas not used for recreation.

B : Green fodder, olives, dates, peanuts. Peeled fruits such as almonds, citrus, nuts, bananas. Ornamental trees and recreational forest.

C : Fruits (irrigation under the trees and should be stopped two weeks before picking time). Vegetables with peels, eaten often after cooking or for conserving industry. Parks, golf courses, lawns (Irrigation area closed to public 24 hours after irrigation).

D : Unrestricted irrigation.

Appendix VIII: Subsurface Flow Wetland Design

The design of every treatment system, and wetlands are no exception, is a circular process, in which earlier stages have to be checked continuously. Designing a subsurface flow consists of several stages (see figure 29):

1. Determination of design parameters: this has been done in §4.1;
2. The choice of the wetland characteristics (plants, media, and lining);
3. Sizing of the wetland, based on pollutant removal;
4. Calculation of water gains of losses, taking into account rain, evapotranspiration and seepage;
5. Hydraulic design of the cross-sectional area of the wetland using Darcy's Law;
6. Calculation of performance of the wetland, and control of design;
7. Layout of the wetland and choice and design of structures;

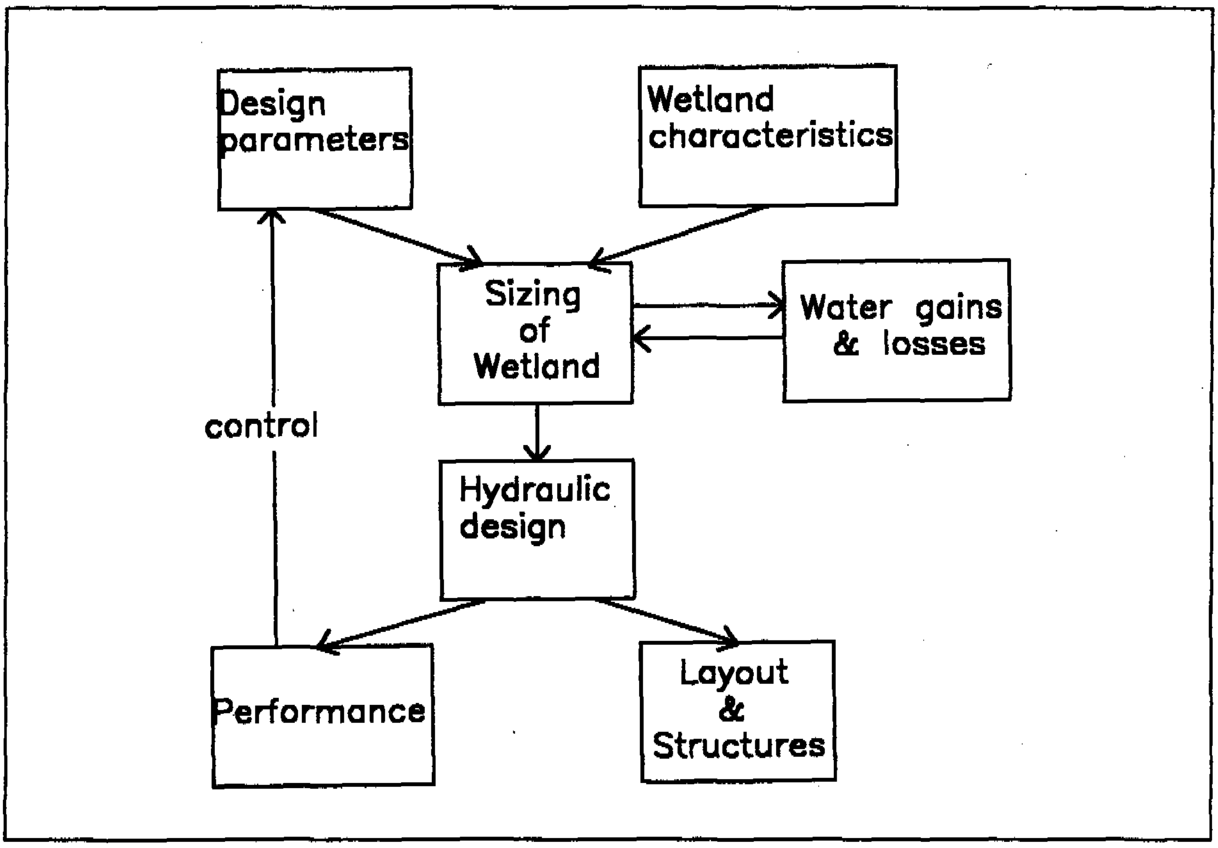


Figure 29: Design structure for subsurface flow wetlands

1. Design concepts

Although research and demonstration projects have shown that wetlands can provide effective wastewater treatment, available information on their performance is often sketchy, at times contradictory and inadequate for optimizing process variables. A survey of all 150 wetlands systems in use in the United States, performed by the U.S. Environmental Protection Agency in 1991, indicated that there was no consensus regarding basic hydraulic and engineering design criteria, system configuration, or any other aspect, such as type of vegetation, size and type of media, or pretreatment (Hammer and Bastian, 1989; Brown and Reed, 1994).

The sizing of the wetland is subject to an academic discussion: there is not yet a consensus among experts as to which approach is the most suitable approach for designing constructed wetlands. Existing design guidelines normally are black box models, in which no separate actions are distinguished, but the result of the total concept is regarded. A number of different procedures for the design exist: design based on analysis of operating systems, design based on areal loading and design based on attached-growth wastewater treatment processes (Reed et al., 1995).

Design based on analysis of performance data from operating systems and the prediction of system performance of the wetland to be constructed by multiple regression methods is only valid if enough experience with wetlands operating under similar circumstances does exist. Data on the functioning of wetlands in tropical areas are even more scarce than on those functioning in moderate climates, as most wetlands in operation are situated in the USA and Europe (Noller et al., 1994; Reed et al., 1995). This approach is therefore not practicable in Cali.

The areal loading approach is similar to that used for land treatment systems. The validity of this method is limited: wastewater is not applied uniformly over the wetland, it does not take into account the depth of the wetland, and it does not recognize the influence of temperature on system performance (Reed et al., 1995). Another drawback of the areal loading approach is that it presumes set effluent concentrations for discharge into rivers. This is a far lower BOD, TSS and TN concentrations than are allowed for agricultural reuse. Loading rates of a system for reuse will therefore be considerably higher than literature values.

A first-order plug flow model assumes that the biological reactions which occur in wetlands are similar to those describing the same type of reactions in other attached-growth wastewater treatment processes. Although a wetland is not an ideal flow reactor and the actual flow regime is probably somewhere between plugflow and complete-mix, there are insufficient data available for the definition of a generally applicable and reasonably easy-to-use design model which takes into account the complicated flow regime in wetlands. Kadlec (1993) states that flow pattern in a gravel bed is not close to plugflow and a more appropriate model would be a dead zone interchanging water with stirred tanks in series comprising the remaining void volume. The plug-flow design method is however the base for the only more or less complete, set of design criteria (Reed, 1990; further developed in Reed et al. 1995; cited in Metcalf and Eddy, 1991 and Kadlec, 1989; cited in Watson et al., 1989).

The plug-flow design method is the simplest adequate design method available. The used design guidelines follow a plugflow design approach and combine the routines developed by Reed and Kadlec. The design is based on BOD and nitrogen removal.

2. Characterization of Wetland

Choice of Macrophyte Species

Wetland plants are generally considered an essential part of a subsurface flow wetland system. The presence of macrophytes stabilizes the bed surface and prevents the formation of erosion channels. By releasing oxygen into the rhizosphere from their roots, aquatic macrophytes create aerobic conditions in the water-satured wetland basin. The roots of macrophytes further provide a huge surface area for attached microbial growth (Brix, 1994; Chamber and McComb, 1994; Reed et al., 1995). Sapkota and Bavor (1994) state that wetland plants are not an indispensable part of subsurface wetland and advocate unplanted, horizontal flow gravel-based systems.

Additional functions not related to the treatment of wastewater have to be taken into account when choosing a species. Nice looking plants (like Yellow Flags and Canna-lilies) make sewage treatment systems aesthetically pleasing. Some species yield useful products (Brix, 1994).

Wetlands plants are morphologically adapted to growing in a water-satured sediment by virtue of large internal air spaces for transportation of oxygen to roots and rhizomes. In order to evaluate suitability of a species for wetland vegetation, the following consideration should be taken into account:

- The species should be able to grow in a gravel bed, fed with nutrient-rich wastewater: the roots of the species should be able to transport oxygen to their rootzone and therefore be able to grow in soil with high freatic levels (±15 cm below ground level) year-round;
- The root system of the species should not decrease the hydraulic conductivity of the gravel bad to such a level that the design discharge cannot flow through the bed and the filter is clogged;
- The root penetration of the species should be quite deep in order to guarantee a deep aerobic treatment zone.

In order to make the system socio-economically interesting the vegetation should:

- provide a useful need, which can be harvested from the wetland regularly, but is not used for human consumption.

Chambers and McComb (1994) list the following factors governing the choice of a plant species:

- There should be no legal or other restrictions against use of the plant in the area;
- Propagation techniques must be available;
- The species should be robust in habit.

As there are no legal restrictions against wetlands plants in Colombia, virtually all local wetlands plants may be used in subsurface flow wetlands systems.

Most constructed wetlands are marsh wetland, although some constructed forest wetlands do exist. The most frequently used species used in wetlands are cattails (*Typha* sp.), reed (*Phragmites Communis*), rushes (*Junces* sp.), bulrushes (*Scurpus* sp.) and sedges (*Carex* sp.). All those species do meet all requirements stated above, only the local availability of source material and propagation techniques should be investigated. Bulrush and cattails are mostly used in the USA, while the use of reed is more popular in Europe. All species can be used for handicrafts, thatching and furniture production.

The root penetration of the macrophyte is the major parameter in the selection of a wetland species: a deeper rootzone, means a deeper flow zone, which means a smaller surface area, with all the advantages mentioned earlier. Field experience on the depth of root penetration is limited to bulrushes, reeds and cattails (Watson et al., 1989). Cattails and rushes are not recommended for use in subsurface flow wetlands because of their shallow root system. The only two species suitable for a full-scale subsurface flow wetland system in Colombia are reeds and bulrushes.

In Cali region some small wetlands are to be constructed using bulrushes, one by CINARA to serve a community of 300 people in La Oragine, just South of Cali, and some one household wetlands constructed by CIPAV. In the near future source material and some experience on propagation techniques, the functioning of this species in wetlands, and use of yielded products will be locally available. Bulrushes are therefore selected to be planted in the wetland.

Table 51: Root penetration of macrophytes

Species name	Latin name	root penetration [cm]
reed	*Phragmites communis*	60 - 76
bulrush	*Scirpus sp.*	76
rushes	*Juncus sp.*	30
cattails	*Typha sp.*	30
bamboo guadua	*Guadua amflexifolia*	100-200
bamboo	*Bambuseae sp.*	30-40

An interesting idea is to use guadua or other types of bamboo as wetland vegetation. Bamboo is a popular local material for handicrafts (bamboo and guaduilla), construction (guadua) and furniture production. It is not clear whether guadua and other bamboo species can grow in gravel beds fed with wastewater. Although bamboo and guadua normally grow in locations near rivers and streams where soils are wet, their root penetration is said to be limited by high freatic levels, and guadua is said to be unfit to grow in fully satured soils, as their root will suffer from anaerobic circumstances (Cruz Rios, 1994). Bamboo species generally prefer well-drained sandy loam to loamy-clay soils, although it can grow on vastly different types of soil; guadua grows best in well-drained soils (Liese, 1985; Castaño, 1993). Agronomic engineers of the Colombian National Institute of Bamboo and Guadua said bamboo is not fit to grow in satured soils. There are some species of bamboo (for instance *Guadua Amflexifolia,* which grows in swamps flooded continuously in Campeche province of Mexico and also grows in the North of Colombia.

Bamboos, especially guadua, have a very big, strong root system. There are no data on the effect of the root system on hydraulic conductivity of the gravel bed. The big roots of guadua could cause erosion channels. Bamboo root penetration is 30-40 cm, guadua roots penetrate up till 200 cm into the soil.

The scarce information on the growing conditions guadua indicate inconsistent data on the suitability of the species for subsurface flow wetlands. Bamboo species are, based on the present knowledge, not suited for the use as wetland vegetation in a full-scale project, as they are no wetland species and will probably not survive in fully satured soils. After further study it will be interesting, however, to try species of guadua or bamboo, which have shown ability to live in wetlands for small-scale projects which are monitored closely.

Choice of Media

Coarsely textured materials, such as gravel, are believed to be the only suitable media for subsurface flow wetlands. Common gravel media include washed and sized crushed limestone and river gravel (Steiner, 1989). In the past soil was used as a media, and plant roots and rhizomes were supposed to maintain the hydraulic conductivity. This system, referred to as the root-zone system, have not showed the desired treatment effectiveness and now function as undersize free water surface flow wetlands as a result of clogging and low hydraulic conductivities.

A subsurface wetland typically contains 0.3 m to 0.7 m of the medium. This layer is sometimes overlain with a layer of fine gravel 76-150 mm deep, which serves as an initial rooting media for the vegetation and is maintained in dry condition during normal operation (Reed et al., 1995).

Gravel and crushed rock are available locally in Cali; gravel is very expensive. Crushed rock of medium size (D= 10 mm) is selected here to serve as media for the wetland.

Lining

On permeable soils prevention of groundwater contamination and water losses can require installation of an impermeable liner below the media. The basic infiltration rate of the Juanchito soil at the wetland site is 4.6 mm/hour, Mara et al. (1992) advise lining if the basic infiltration rate of the soil is more than 0.36 mm/hour. The liner must be strong, thick and smooth to prevent root penetration and attachment. Possible materials are compacted in situ soils (permeability less than 10^{-6} cm/s); bentonite; asphalt; synthetic butyl rubber; or plastic membranes (Steiner, 1989). Geotextile, a sheet material that is available in Columbia, is selected to line the wetland.

3. Design Parameters

The design parameters are the same as those of the duckweed pond system in chapter 4. The design temperature is 25 °C, which equals the design temperature of the Cañaveralejo plant. The design discharge is 475 l/s.

Table 52: design parameters for pre-irrigation system

parameter	symbol	unit	influent	effluent	removal
Discharge	Q	m^3/day	-	41,000 (475 l/s)	-
Temperature	T	°C	25	25	-
BOD	C	mg/l	106	≤50	53%
TSS	C	mg/l	129	≤50	61%
Nitrogen	C	mg/l	26	≤20	23%
Helminth eggs	C	#/l	300	≤1	99.7%
Cadmium	C	mg/l	0.016	0.01	38%

4. Sizing of Wetland based on Removal of BOD and Nitrogen

First-order Plugflow Kinetics

If the wetland is considered to be an attached-growth biological reactor, its performance can be estimated with first-order plug flow kinetics for BOD and nitrogen removal. The basic relationship for steady conditions for plugflow reactors is given by equation 45:

$$\frac{C_e}{C_o} = \exp(-K_T * t) \tag{45}$$

where C_e = effluent pollutant concentration [mg/l];
C_o = influent pollutant concentration [mg/l];
K_T = temperature-dependent first-order reaction rate constant [d^{-1}];
t = hydraulic residence time [d];

The hydraulic residence time in the wetland can be calculated using equation 46:

$$t = \frac{L*W*y*n_p}{Q} \tag{46}$$

where L = length of wetland cell [m];
W = width of wetland cell [m];
y = depth of water in the wetland cell [m];
n_p = porosity of the media [-];
Q = average flow through the wetland [m^3/d];

By combining equations 45 and 46 it is possible to determine the surface area of the wetland:

$$A_s = L*W = \frac{Q*\ln(C_0/C_e)}{K_T*y*n} \tag{47}$$

where A_s = surface area of wetland [m^2];

The value used for K_T in equations 45 and 47 depends on the pollutant requiring removal and on the temperature.

Sizing of Wetland based on BOD-removal
BOD removal in a subsurface flow wetland is performed by the disposition and filtration of settleable organics within the first few meters of the bed, where it undergoes further decomposition. The remaining BOD, in colloidal and dissolved forms, continues to be removed as the wastewater comes in contact with the attached microbial growth in the system. BOD removal in a subsurface flow wetland can be described by the first-order plugflow equations 45 and 47.

Watson et al. (1989) suggest a BOD-removal determination based on the oxygen balance of a wetland. The oxygen demand for BOD-removal should equal the oxygen transmitted by macrophytes into their rootzone. Commonly used emergent plants can transmit for 5 to 45 gram O_2 per day per square meter into their rootzone. By estimating the oxygen demand to be equivalent to that for partial-mix aerated ponds (20 g/m^2/day), the oxygen balance in a subsurface flow wetland can be checked. This method, however, presuppose that biodegradation in a wetland is a completely aerobic process, and depends solely on the input of oxygen through roots. I think this presupposition is incorrect, as substantial removal efficiencies of organic matter can be reached by unplanted gravel filters (Sapkota and Bavor, 1994).

Reed et al. (1995) provide a relationship for the first-order reaction rate constant (K_T) based on temperature and a rate constant at 20°C. Watson et al. (1989) suggest a first-order reaction rate constant, which is related to the porosity of the bed media, and an optimum rate constant for a medium with fully developed root zone. The porosity of the bed, however, is difficult to determine because it will decrease and vary within the bed, due to root density and clogging. In my opinion the more extensive formula will therefore create a false accuracy. The simpler relationship given by Reed et al. (equation 48) will hence be used.

$$K_T = K_{20} * 1.06^{(T-20)} = 1.104 * 1.06^{(T-20)} \qquad \textbf{(48)}$$

where: T = temperature [°C]
K_{20} = first-order reaction rate constant at 20°C [d^{-1}]

The surface area required to guarantee BOD-removal is 90,100 m^2, which equals 9 ha, when applying first-order plugflow equations with the design data (C_0=106 mg/l, C_e= 50 mg/l and T = 25°C).

Sizing of Wetland based on Nitrogen Removal
Removal of nitrogen can be performed by a combination of nitrification and denitrification and by nitrogen uptake of macrophytes. The conditions in a subsurface wetland, which will be aerobic nearby the roots of the macrophytes and anaerobic in other parts, create an environment for both nitrification and denitrification.

Nitrification is the conversion of ammonia into nitrate, and can be described by formula :

$$NH_4^+ + 2\ O_2 \rightarrow NO_3^- + H_2O + 2\ H^+ \qquad \textbf{(49)}$$

Nitrification in subsurface wetlands depends on the availability of oxygen in the rootzone: removal of 1 g of ammonia requires 4.6 g of oxygen. Full scale subsurface flow wetlands have not shown the nitrogen removal capability found in early pilot scale research and the availability of oxygen is subject to a fierce academic discussion and there is no consensus on the oxygen transfer efficiency of the various plant species. Most current design guidance are therefore very conservative (Brown, 1994). It is prudent to assume that all of the Kjeldahl nitrogen (TKN) entering the system will eventually be converted to ammonia. One way of calculating nitrification is using the first-order plugflow equations

45 and 47, K_T now being a temperature-dependent nitrification rate constant depending on the fraction of the depth actually occupied by rootzone (Reed et al., 1995).

$$K_T = K_{NH}*(1.048)^{(T-20)} \tag{50}$$

$$K_{NH} = 0.01854 + 0.3922*(rz)^{2.6077} \tag{51}$$

where: K_{NH} = nitrification rate constant at 20°C [d^{-1}];
rz = percent of bed depth occupied by rootzone, a decimal fraction between 0 and 1;

For new wetland in which no measurements of the rootzone fraction (rz) can be taken, it is not acceptable to assume that the rootzone will automatically occupy the entire bed volume. The climate conditions in the Valle de Cauca, however, favors deep root penetration though, and an estimate of 0.8 is used for the rootzone fraction. The temperature used is 25°C. This results in the following equation for nitrification:

$$A_s = \frac{Q*\ln(C_0/C_e)}{0.300*y*n} \tag{52}$$

Equation 52 only accounts for nitrification, the conversion of ammonia to nitrate, and predict the area required for a given level of conversion. Actual removal of nitrogen is the requirement of the wetland, and it is necessary to consider the denitrification and size of the wetland accordingly. Denitrification is the conversion of nitrate into oxygen and nitrogen. Denitrification requires anaerobic circumstances, these will occur in parts of the wetland. Denitrification can be described as:

$$2\ NO_3^- + 2\ H^+ \rightarrow N_2 \uparrow + H_2O + 5\ O \tag{53}$$

Once again the first-order plugflow equations 45 and 47 are used for describing denitrification. K_T for denitrification can be described using equation 54, 54:

$$K_T = 1.000*(1.15)^{(T-20)} \tag{54}$$

Given a water temperature of 25°C, this results in the following first-order plugflow equation:

$$\frac{C_e}{C_o} = \exp(-2.01*t) \tag{55}$$

Total Kjeldahl nitrogen removal is the sum of the first-order plug flow equations for nitrification and denitrification. The surface area required is determined by an iterative procedure, taking into account nitrification and denitrification. The surface area of the wetland given the design parameters (C_0 = 26 mg/l and C_e = 20 mg/l) is 19,4 ha.

Nitrogen uptake by macrophytes is normally not taken into account when designing wetland. One reason for this is that in most wetlands the macrophytes are not regularly yielded, and therefore removal is limited. Besides data on nitrogen removal are not readily available. Brix (1994) estimates between 1000 and 2500 kg per hectare per year. A 1500 kg/ha*year removal would equal a 1.97 mg/l nitrogen removal.

Determination of Surface Area
The surface area required for BOD-removal is 9.0 ha, the surface area for nitrogen removal is 19.4 ha. The surface area of the wetland will therefore be 19.4 ha.

The hydraulic loading rate of the wetland is 0.21 m/day (see equation 56).

$$HLR = \frac{Q}{A_s} \qquad (56)$$

where: HLR =hydraulic loading rate [m/day];

5. Hydraulic Design

Evaporation and Rainfall
Subsurface flow wetland systems interact with the atmosphere via rainfall and evapotranspiration, because of large surface areas, outdoor locations, and long retention times. Evapotranspiration slows the water and increases hydraulic residence time, but on the other hand concentrates the pollutants in the wastewater. Rainfall has the opposite effect, it shortens hydraulic residence time and dilutes pollutant concentrations.

Flow through a wetland system is augmented by precipitation and (negatively) by evapotranspiration. It is necessary to determine the average flow of the wetland, as water losses via seepage and evapotranspiration and gains via seepage and precipitation can be considerable. The second effect of evaporation and rainfall, the dilution of wastewater by rainfall or concentration by evapotranspiration and seepage is nowhere accounted for in literature. In order to estimate the gains and losses in the wetland the surface area of the wetland has to be estimated, evapotranspiration, seepage and precipitation has to be calculated. Because the wetland bed is lined, seepage will only occur at inlet and outlet structures, and therefore is not a function of the surface area of the wetland. There are two ways of estimating water gains from evaporation and rainfall: an average flow approach which presumes linear evapotranspiration and rainfall (Reed et al., 1995) and the more complex actual retention time approximation, presented by Kadlec (1989). As only a very small part of the influent flow is lost due to the water deficit between evaporation and precipitation, both methods are similar and the simpler average approach is used here.

$$\Delta Q = A_s*(P-k_c*E_0)-S \qquad (57)$$

$$Q_e = Q_0 + \Delta Q \qquad (58)$$

where: ΔQ = flow gain [m^3/d];
P = average precipitation [m/month];
E_0 = evaporation [m/month];
k_c = crop coefficient [-];
S = seepage [m^3/d];

The crop factor (k_c) for a subsurface wetland can be estimated as 0.8 (Kadlec, 1989; Reed et al., 1995). The month in which evaporation is lowest and rainfall is highest, the wettest month, is the measure for water gains and losses. The wettest month in the area is April, evaporation (E_t) is 4.3 mm/day and average rainfall (P) is 2.8 mm/month (based on Rio Cauca station, appendix IV).

The flow gain is 133 m^3/day, which is 0.3% of the daily flow (equation 57). The water gain is so small that it is not taken into account for the design.

Cross-Sectional Area

One of the mayor problems of subsurface flow wetlands in operation is the occurrence of surface flow, which results in short circuiting and reduced retention times. This problem has been primarily blamed on plugging of the void spaces in within the media by wastewater solids, biomass and plant roots. Further investigation has revealed, however, that in many of the subsurface systems, inadequately hydraulic design wast the cause of the surface flow. In early design of wetland using soil as media, design is based on the assumption that the hydraulic conductivity would rise after the establishment, it is now generally accepted that the conductivity decreases (Brown, 1994).

Proper hydraulic design can solve clogging problems. Based on this surface area the width of wetland can be determined based on hydraulic consideration: in order to prevent clogging, the wetland has to have a minimum cross-sectional area. The cross-sectional area is the product of the width of the wetland (W) and the depth of the wetland (y). In order to determine the cross-sectional area of the wetland, the flow through the wetland is described using Darcy's law.

Darcy's law (equation 59) describes the flow regime in a porous media and is generally accepted for design of subsurface flow wetlands, with a lateral flow in fully saturated, fine grained soils, sands and gravels (Kadlec, 1989).

$$Q = k_s * A_c * s \qquad \textbf{(59)}$$

where: k_s = hydraulic conductivity of a unit area of the wetland perpendicular to the flow direction $[m^3/m^2/d]$;
A_c = total cross-sectional area perpendicular to flow $[m^2]$;
s = hydraulic gradient, or "slope" of the water surface in the flow system [m/m].

Measured values and design estimates of hydraulic conductivity vary widely. The lowest value measured in a gravel bed in England is 30 m/d (Watson and Hobson, 1989). Kadlec and Watson (1993) found that hydraulic conductivity of the inlet zone of a coarse gravel bed had been lowered substantially by the mud formation in the first year of operation; after that conductivity did not change. Bed average conductivities were lower in the plugged zone by a factor of 10. Appropriate values for hydraulic conductivity are likely between that of clean gravel (864 m/d) and the minimum measured in England. For design purposes the USA Environmental Protection Agency recommends that only 10% of the clean media hydraulic conductivity should be used in calculations (cited in Brown, 1994). Reed et al. (1995) recommend to use one third of the actual hydraulic conductivity of the clear media for design purposes. Watson and Hobson (1989) state that a permeability of 260 $m^3/m^2/d$ is stabilized in the long-term for gravel-bed systems with roots, regardless the initial conductivity. This value will be used here.

By introducing a factor which indicates the increment in depth serving as head differential, Darcy's law can be rewritten as:

$$W = \frac{1}{y} * [\frac{Q * A_s}{m * k_s}]^{0.5} \qquad \textbf{(60)}$$

where: m = increment of depth serving as head differential [-] = s*L/y;

The m value will typically be between 5 and 20% of the head available (which equals the water depth y), to provide a large safety factor against potential clogging, viscosity effect and other contingencies that may be unknown at the time of design. A value of 0.2 is used here. Given the bed depth (y=0.6m), average discharge (Q=41,000 m^3/d), and surface area of the wetland (19.4 ha), the width (W) can be calculated using equation 60, and is 20,472 m. Length is calculated by dividing the surface area by the width, and is 7.5 m.

Darcy's law assumes laminar flow. In order to check if the subsurface flow in the wetland is laminar the Reynolds number can be calculated using equation 61. For laminar flow the Reynolds number has to be less than 10.

$$N_R = \frac{v * D_m}{\tau_w} \tag{61}$$

where: N_R = Reynolds number [-];
v = Darcy's velocity [m/s] = $k_s * s$;
D_m = Diameter of void spaces in media, taken as the equal size of the media [m];
τ_w = kinematic viscosity of water [m^2/s];

The Reynolds number is 1.86, so the flow in the wetland is laminar.

6. Layout of Wetland

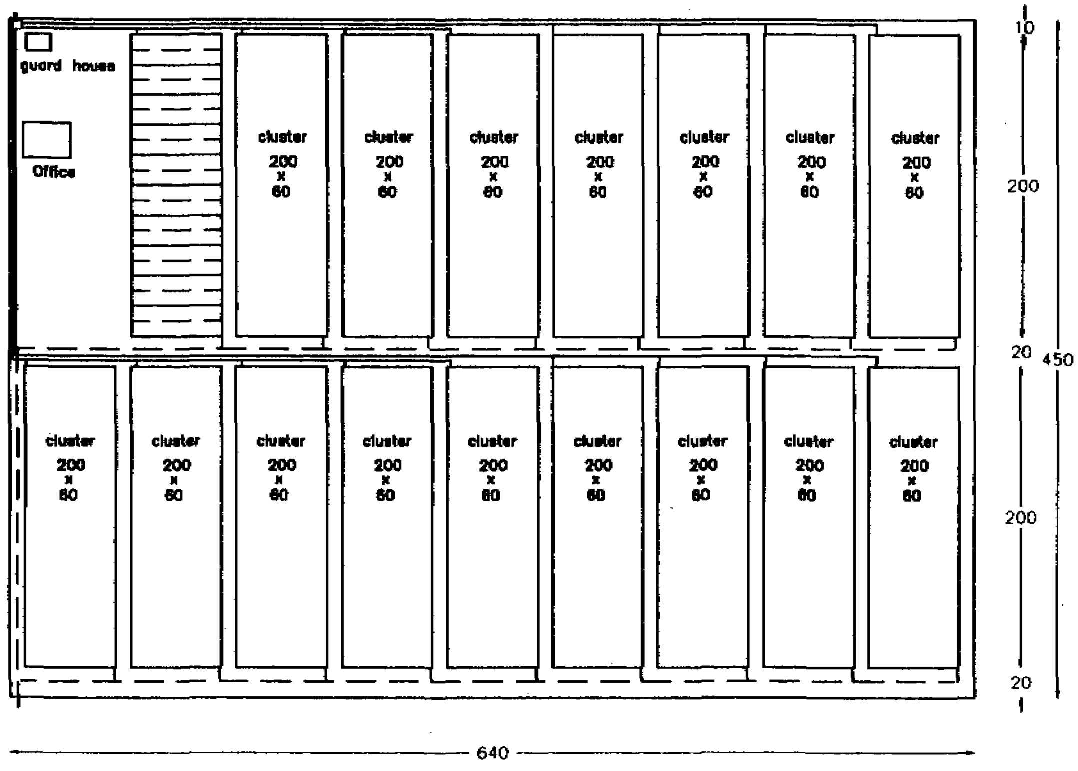
In the above the depth, the surface area, the width and the length of the wetland are determined. In order to guarantee enough freedom in operation of the wetland, the wetland has to be divided in several parallel cells. Grouping is shown in figure 30. Storage is not designed in this study.

The strip of land between the River Cauca and the River Fraile 450m wide, the inlet of the wastewater is at the South-Western corner, near the railway bridge, the outlet at the South-Eastern corner.

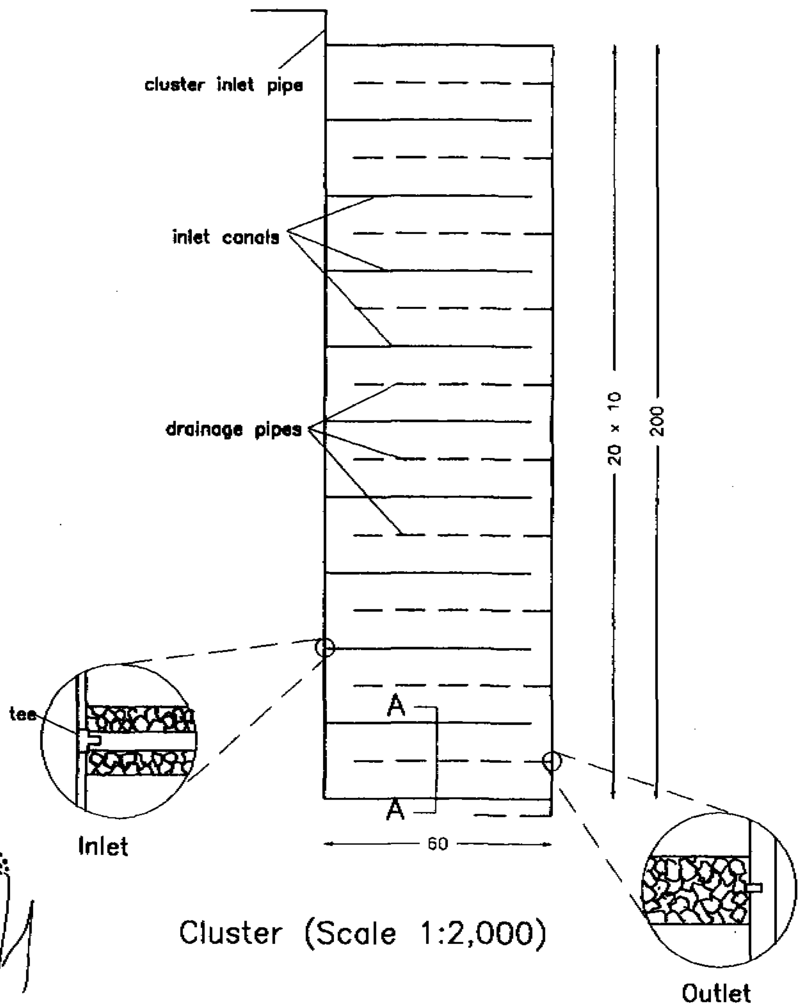
In order to increase ease of operation of the wetland, the wetland will be divided in several parallel cells.

The wetland will divided into 170 wetland cells with a central 50 long inlet canal border by two strips of 10m long planted gravel bed. A level surface allows flooding for weed control and a minimum slope for the base allows the water to pass through the bed (Cooper, 1993).

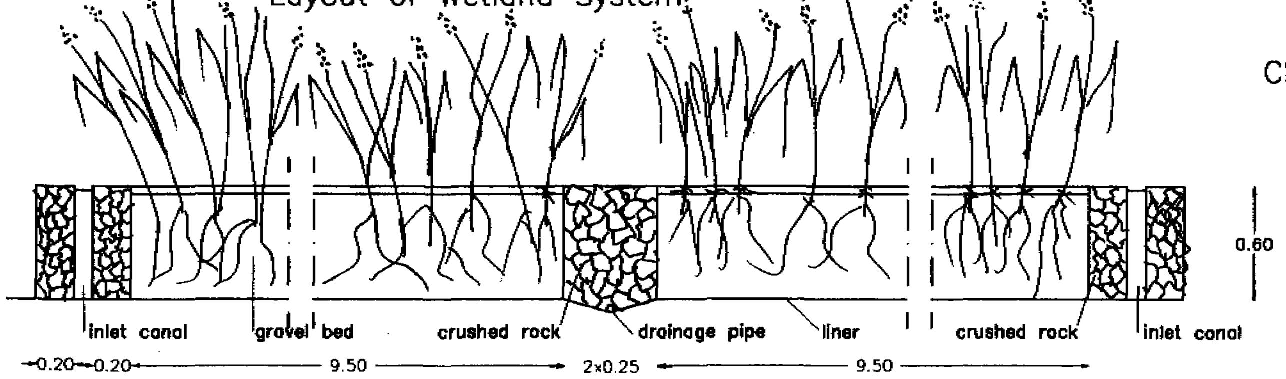
The wetland cells are grouped in 17 parallel units of 10 cells, each of which is served by a secondary pipeline. The units are clustered in four groups, each served by one primary pipeline.



Layout of Wetland System



Cluster (Scale 1:2,000)



Cross–Section A – A

Figure 33	
Subsurface Flow Wetland System	
Canaveralejo Reuse Scheme	
Meike van Ginneken	
Date: 8 – 1996	Scale 1:5,000

7. Structures and Piping

Subsurface flow wetlands require uniform flow conditions through each cell to achieve the expected performance. A number of different inlet designs have been used.

A small to moderate-sized system can have manifold pipes, extending the full width of the cell, for both inlet and outlets. In larger systems concrete inlet and outlet structures can be used.

An above-surface inlet manifold provides access for adjustment, cleaning, and control. This manifold typically consists of suitable-sized pipes 100-200 mm in diameter, with a "tee" placed in the line every 3 meter. These "tees" are attached to the line with O-rings and area adjustable: the operator can move each "tee" through a vertical arc and thereby visually adjust and equalize the flow from each. Another alternative inlet arrangement is to pipe the feel into a "V-shaped" wedge formed by the sloping side wall and the vertical wall of the gabion. Using a weir for flow distribution is not recommended, since these are expensive to construct and maldistribution of flow is often caused by material collecting on the edges (Cooper, 1993; Reed, 1995).

The effluent manifold for a subsurface flow wetland is typically a perforated pipe laid on the bottom of the bed at the outlet end of the cell. In some cases this outlet manifold is laid in a shallow, rock filled trench slightly below the bottom of the wetland cell to allow for complete drainage. Larger systems typically have concrete outlet structures containing a weir or similar device; the devices should be adjustable, to permit control of the water level in the cell (Reed et al, 1995).

The design of inlet and outlet structures is based on local experience and varies from country to country. Some examples are shown in the following drawings, which are derived from the following sources: Watson et al.(1989), Davies et al. (1993), Lekven et al.(1993), Cooper (1993), Brix (1993), and Reed et al. (1995).

8. Performance of Wetland

BOD removal

The BOD-concentration in the effluent is 9 mg/l. This concentration is obtained from the first-order plugflow equations 45 and 47, using the reaction factor given in equation 48, and the surface area calculated for nitrogen removal.

TSS removal

The removal of suspended solids (TSS) is not likely to be the limiting design parameter for sizing the wetland, since TSS removal is vary rapid as compared to removal of either BOD or nitrogen (Reed et al., 1995). The effluent TSS concentration of the wetland area is 14 mg/ (equation 62).

$$C_e = C_0 * (0.1058 + 1.1 * 10^{-5} * HLR) \qquad \textbf{(62)}$$

Pathogen removal

A four log removal of faecal coliforms results in an effluent concentration of 300 coli per liter. Complete removal of helminth eggs is achieved. Pathogen removal is based on removal in sand filtration beds and waste stabilization ponds. Removal of pathogens in wetlands is based on filtration, sedimentation and die-off. The information on pathogens removal in wetlands is very limited, Rivera et al. (1995) observed varying pathogen removals, put the number of helminth eggs was below levels of detection in subsurface flow wetland effluent.

Reed et al. (1995) suggest using the pathogen removal equations of waste stabilization ponds. These equations describe sedimentation and die-off, and do not take into account the filtration taking place in wetlands. Actual removal should be more effective due to the additional filtration provided by the roots and media.

The removal of pollutants in subsurface flow wetland systems is similar to that in slow sand filtration. The wetland described here has a hydraulic loading rate of 0.29 $m^3/m^2/day$, slow sand filtration normally has a hydraulic loading rate between 2 and 5 $m^3/m^2/day$. At this rate four log units of removal may be expected from a well-run unit. Complete helminth eggs removal has been recorded (Feachem et al., 1983; IRC, 1987). The hydraulic loading rate of wetlands is considerably lower and process are similar, it is appropriate to assume similar or slightly higher removal efficiencies.

Salinity
Salinity in wetlands is changed by concentration water losses through evaporation and seepage, dilution by water gains through rainfall, and removal by sedimentation of saline particles. The net removal in the Ginebra lagoons is zero, as a wetland has a considerable shorter hydraulic retention time, there will probably be a net removal of a minor part of the salts. A prudent presumption, however, is that no removal will take place. Effluent electric conductivity will therefore equal influent electric conductivity (0.7 dS/m).

Trace Elements Removal
Trace elements removal is estimated to be 90%. The major mechanisms responsible for trace elements removal in subsurface flow wetlands are precipitation and adsorption interactions with the organic benthic layer. Plant uptake accounts for less than 1% of heavy metal uptake. Reed et al. state excellent removal (>95%) of trace elements can be sustained over the long term during the design life time of wetland systems. Trace elements will accumulate in the wetland system, but at the concentrations normally found in wastewaters they should not represent a long-term threat to the habitat values of the site or to alternative uses.

water quality of effluent

			desired effluent	influent	subsurface flow wetland	
					effluent	removal
general	discharge	l/s	450	500	450	-
	pH	units	6.5-8.4	6,7	6,7	-
	temperature	°C	-	25	25	-
gross pollutants	BOD	mg/l	50	106	9	92%
	TSS	mg/l	50	129	14	89%
nutrients	total nitrogen	mg N/l	20,0	26	20	23%
pathogens	faecal coliforms	#/100ml	-	3e+07	3e+03	100%
	helminth eggs	#/liter	1	300	1	100%
salinity	ECw	dS/m	1,7	0,7	0,7	0%
	SAR		0-5		0-5	0%
specific ions	Chloride	me/l	3,0	1,6	1,6	0%
trace elements	cadmium	mg Cd/l	0,010	0,016	0,001	90%
	copper	mg Cu/l	0,200	0,013	0,001	90%
	nickel	mg Ni/l	0,200	0,115	0,012	90%
	lead	mg Pb/l	5,000	0,028	0,000	90%
	zinc	mg Zn/l	2,000	0,092	0,006	90%

9. Costs and Benefits

Investments costs of a subsurface flow wetland (US$4,400,000) are 50% higher than those of a pond system (US$2,900,0000). This is mainly due to the high price of gravel and the larger surface area occupied by the wetland. Table 53 presents a rough estimates of the investment costs of a subsurface flow wetland. The integration of storage reservoir, and resulting elevation of the complete system is not taken into account. The costs for structures, piping, and facilities are presumed to be the same as those of the pond system.

Table 53: Investments of subsurface flow wetland

item	unit	unit cost	amount	price
Land acquisition	m2	$10	230,000	$2,300,000
Building of wetlands				
excavation	m3	$3	50,000	$150,000
liner	m2	$1	160,000	$170,000
clearing and grubbing	ha	$493	20	$10,000
gravel	m3	$14	100,000	$1,400,000
Structures/piping				$180,000
Facilities				$170,000
TOTAL				$4,400,000

Appendix IX: Prices

inflation rate %	21%			exchange rate	1048 col$ = 1US$		
item	**unit**	**price 5-96**	**price 5-96**	**quoted price**	**year**	**month**	**source**
		US$	**Col$**	**col$**			
land							
land	m2	10	10488	10000	1996	1	various
rent of land	month/ha	38	39971	31000	1994	12	URPA
energy	kwh	0	75	72	1996	1	CINARA
building materials							
excavation	m3	3	3605	3548	1996	3	EMCALI
fine sand	m3	9	9605	9158	1996	1	Construdata
medium sand	m3	9	9605	9158	1996	1	Construdata
coarse sand	m3	9	9605	9158	1996	1	Construdata
gravel (incl transport)	m3	14	14940	14245	1996	1	Construdata
cement (grey)	kg	0	210	200	1996	1	Construdata
concrete (1500 psi)	m3	81	84429	80500	1996	1	Construdata
concrete (2000 psi)	m3	82	85793	81800	1996	1	Construdata
concrete (2500 psi)	m3	85	88729	84600	1996	1	Construdata
crushed rock	m3	6	6403	6105	1996	1	Construdata
steel pipe 28"	m	97	101134	96427	1996	1	Construdata
steel pipe 18"	m	57	59863	57077	1996	1	Construdata
pipe 24" concrete (incl installation)	m	57	59332	58397	1996	3	EMCALI
pipe 21" concrete (incl installation)	m	47	49046	48273	1996	3	EMCALI
pipe 18" concrete (incl installation)	m	35	36530	35954	1996	3	EMCALI
pipe 15" concrete (incl installation)	m	27	28421	27973	1996	3	EMCALI
Geotextile 1600	m2	1	905	863	1996	1	geotextile pricelist
geotextile 1600 (incl installation)	m2	1	1417	1395	1996	3	EMCALI
geotextile 2500	m2	1	1004	957	1996	1	geotextiel pricelist
geotextile 2500 (incl installation)	m2	2	1925	1895	1996	3	EMCALI
office building	#	140874	147636071	150000000	1996	5	based on design Nitogoi
guard house	#	4696	4921202	5000000	1996	5	based on design Nitogoi
pump building	#	93916	98424047	100000000	1996	5	based on design Nitogoi
labor							
min.salary	day	5	5374	3500	1994	1	URPA/CONE
foreman/supervisor	month	921	964904	920000	1996	1	Construdata
mechanical engineer	month	701	734166	700000	1996	1	Construdata
laboratory technician	month	400	419524	400000	1996	1	Construdata
assistant foreman	month	590	618797	590000	1996	1	Construdata

laborers	month	213	223396	213000	1996	1	Construdata
drivers	month	200	209762	200000	1996	1	Construdata
watchmen	month	200	209762	200000	1996	1	Construdata
agricultural laborers	day	5	5663	3300	1993	6	CONE
agricultural/irrigation							
yielded cane	ton	-23	-24498	-19000	1994	12	CONE
Urea	kg	0	286	186	1994	1	URPA
KCl	kg	0	298	194	1994	1	URPA
Soil analysis	#	10	10720	2649	1988	12	URPA
mechanized application	day	17	17380	4295	1988	12	CVC
biologic control	inch	0	100	65	1994	1	URPA
reservoirs (incl in&outlets)	m3	2	2000	2000	1996	4	CENICAÑA
primary canals	m	17	18000	18000	1996	4	CENICAÑA
tertiary canals	m	7	7342	7000	1996	1	CENICAÑA
flexitubes (incl. mouths)	m	1	1090	710	1994	1	Benitez, 1994
secondary canals	m	13	14000	14000	1996	4	CENICAÑA
surface water	m3	0	27	25	1995	12	CENICAÑA
groundwater	m3	0	80	75	1995	12	CENICAÑA
drain 65mm	m	1	1482	1413	1996	1	Construdata
clearing and grubbing	ha/year	493	516529	186884	1990	12	CENICAÑA
soil preparation + sowing	ha/year	383	401559	145287	1990	12	CENICAÑA
initial practices	ha/year	25	25923	9379	1990	12	CENICAÑA
weed control	ha/year	60	63094	22828	1990	12	CENICAÑA
fertilization	ha/year	125	130887	47356	1990	12	CENICAÑA
mechanized labor	ha/year	35	36956	13371	1990	12	CENICAÑA
irrigation	ha/year	455	476631	172449	1990	12	CENICAÑA
drainage	ha/year	106	110948	40142	1990	12	CENICAÑA
miscellaneous	ha/year	35	36970	13376	1990	12	CENICAÑA
fixed costs	ha/year	60	62837	22735	1990	12	CENICAÑA
maintenance reservoirs	ha/year	4	4194	1517,25	1990	12	CENICAÑA
maintenance materials	ha	1	1352	489	1990	12	CENICAÑA
Chemical canal maintenance	ha	4	3983	1441	1990	12	CENICAÑA
treatment							
bamboo	m2*year	0	-60	-32	1993	1	Castaño, 1993
duckweed	ton	-12	-12378	-10560	1995	6	CIPAV
desludger hire	day	2502	2622022	2500000	1996	1	CINARA
secondary treatment	m3/s	$17*10^6$	$18*10^9$	$9*10^9$	1992	6	EMCALI
O&M sec.treatment	m3/year	0	8	4	1992	6	EMCALI
sludge management	m3	15	15356	10000	1994	1	EMCALI

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