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LIST OF ABBREVIATIONS

a	year
aET	actual evapo-transpiration
bgl.	below ground level
BMBF	Federal German Ministry for Education and Research
BMU	Federal German Environmental Ministry
BMZ	Federal German Ministry for Economic Cooperation
BOD_5	Biological Oxygen Demand in Five Days
cap	Capita
cm	Centimetre
°C	Degree Celsius
d	Day
DN	Calculated diameter
GB	Green Belt
GTZ	Deutsche Gesellschaft für Technische Zusammenarbeit (German Technical Cooperation)
h	Hours
ha	Hectare
kg	Kilogram
km	Kilometre
km^2	Square Kilometre
l	Litre
lps.	Litre per Second
M	Million
m	Metre
m^2	Square Metre
m^3	Cubic Metre
max	Maximum
mg	Milligram
µg	Microgram
min	Minimum
mm	Millimetre
NE	North-East
MSL	Mean Sea Level
P	Person

PE	Population Equivalent
PET	Potential Evapo-transpiration
RWHS	Rainwater Harvesting System
RZTP	Root Zone Treatment Plant
s	Second
SW	South-West
tot	Total
TP	Treatment Plant
WW	Water Works
WWTP	Wastewater Treatment Plant

1 INTRODUCTION

1.1 Background

On 30 -31 October 2000 in Bonn, Germany, the International Symposium on "ECOSAN – closing the loop in wastewater management and sanitation" was held in co-operation with the BMU, BMBF, BMZ and GTZ. In the opening ceremony Dr. Usch Eid Parliamentary State Secretary
Federal Ministry for Economic Cooperation and Development
stated:

"Finding solutions to the world water crisis is probably the most challenging task the international community is facing today. A change in perception as well as concrete action is required to achieve sustainable and integrated water management.

Across the world there are 1.3 billion people who have no access to clean water. Twice as many have no adequate sanitation facility.

Children and women in particular lack access to sufficient water, as do small farmers. Ecosystems are damaged or destroyed by over-exploitation of water and by pollution.

Unfortunately, the facts available at present indicate that the situation is likely to become more, rather than less, acute in the future. Population growth implies an increased demand for water. The increasing need of water by private households, industry and agriculture further depletes supplies of clean water. One key problem is the wastage of water due to inefficient use, be it in agriculture or urban water supply. Often, it is because water is free or heavily subsidised that it is wasted in this way. Ultimately, increasing water shortages lead to rising prices and battles over distribution that can even, in some cases, escalate into violent conflicts. These conflicts may arise between individual consumers, groups of consumers, regions or countries.

In the future, water must be used more sparingly and more efficiently. To achieve this, not only do we need a new awareness among users, planners and the authorities but the necessary political decisions will also have to be taken. Managing demand is frequently not only a more sustainable way of dealing with the problem but also cheaper than tapping new sources.

Water has now also become a major issue for international debate. In its Global Environment Outlook (GEO 2000), the United Nations Environment Programme

(UNEP) quite correctly points out that, second only to the danger of climate change, the freshwater crisis is the greatest ecological threat of our times.

This urgent need to take action in the area of freshwater has just recently been emphasised in the United Nations Millennium Declaration adopted by the Millennium Assembly on 8 September 2000. The Millennium Assembly declares that by 2015 the proportion of people who are unable to reach or to afford safe drinking water will be halved and that the unsustainable exploitation of water resources will be stopped by developing water management strategies at regional, national and local levels that promote both equitable process and adequate supplies."

In this symposium Mr. Roland Schertenleib Water and Sanitation for Developing Countries (SANDEC), Swiss Federal Institute for Environmental Science and Technology (EAWAG), presented:

"The Bellagio Principles

A group of 25 experts, drawn from a wide range of international organizations involved in environmental sanitation, both from headquarters offices and the field, met at Bellagio, Italy, from 1-4 February 2000."

"The following principles were proposed as the underpinning basis for a new approach:

1. **Human dignity, quality of life and environmental security at household level should be at the centre of the new approach, which should be responsive and accountable to needs and demands in the local and national setting.**
    - solutions should be tailored to the full spectrum of social, economic, health and environmental concerns.
    - the household and community environment should be protected.
    - the economic opportunities of waste recovery and use should be harnessed.
2. **In line with good governance principles, decision-making should involve participation of all stakeholders, especially the consumers and providers of services.**
    - decision-making at all levels should be based on informed choices.
    - incentives for provision and consumption of services and facilities should be consistent with the overall goal and objective.
    - rights of consumers and providers should be balanced by responsibilities to the wider human community and environment

3. **Waste should be considered a resource, and its management should be holistic and form part of integrated water resources, nutrient flows and waste management processes:**
    - inputs should be reduced so as to promote efficiency and water and environmental security.
    - exports of waste should be minimized to promote efficiency and reduce the spread of pollution.
    - wastewater should be recycled and added to the water budget.
4. **The domain in which environmental sanitation problems are resolved should be kept to minimum practicable size (household, community, town, district, catchment, city) and wastes diluted as little as possible:**
    - waste should be managed as close as possible to its source.
    - water should be minimally used to transport waste.
    - additionally technologies for waste sanitation and re-use should be developed.

1.2 billion people do not have access to safe drinking water.
3 billion people do not have access to proper sanitation.
50 % of all solid waste is uncollected.
No -one knows how many people are flooded out each year and
3 billion people have to survive on less then US$ 2/day."

In the light of this approach the following Auroville Water Management Concept has been presented at this Symposium as a contribution to sustainable urban planning.

Figure 1.1: Study Area Location in India

Figure 1.2: Study Area Location in Tamil Nadu

1.2 The City of Auroville

Auroville was founded as an international township for 50,000 inhabitants. The official inauguration took place on February 28th, 1968, with a formal ceremony around an urn into which earth from 124 countries was placed as a symbol of human unity.
The project received unanimous endorsement at the General Conference of UNESCO in 1966, 1968, 1970 and 1983.
The aims of the city are outlined in its Charter as follows:

1. Auroville belongs to nobody in particular. Auroville belongs to humanity as a whole. But to live in Auroville one must be the willing servitor of the Divine Consciousness.

2. Auroville will be the place of an unending education, of constant progress, and a youth that never ages.

3. Auroville wants to be the bridge between the past and the future. Taking advantage of all discoveries from without and from within, Auroville will boldly spring forwards future realizations.

4. Auroville will be a site of material and spiritual researches for a living embodiment of an actual Human Unity.

The Mother, February 28, 1968

The city of Auroville is planned to cover a circular area with a diameter of 2,5 km, and to be surrounded by a 1,25 km-wide greenbelt of forest and farmland. The city is to be comprised of four zones: Cultural, International, Industrial, and Residential. Parks and green corridors are to be included within all the zones.
The township was established on a plateau with a maximum elevation of 52 m above sea level. It lies both in the eastern coastal part of the South Arcot district of Tamil Nadu and in the north of the Union Territory of Pondicherry.
The geographical centre of the township is located 5 km from the coast of the Bay of Bengal. The township is surrounded by 20 villages with an approximate total

population of 35,000 inhabitants. The closest city is Pondicherry, located at a distance of 12 km from Auroville. The capital of Tamil Nadu, Chennai (Madras), is located approximately 160 km to the north.

The present community of Auroville consists of over 80 settlements of varying size, accommodating a total of about 1,700 inhabitants from 35 nations. The basic infrastructure, consisting of roads, water supply, electricity and telecommunications, has been established for the present population, but needs expanding for the external township.

In 1968, the land was a vast dry open expanse of red earth, scarred by a network of gullies and ravines carved out over the years by the combined effects of monsoon rains, 200 years of deforestation, and poor land management practices. The main focus of the settlers in Auroville was initially to stop the process of erosion, the loss of top soil, and the stormwater runoff. Soil and water conservation programmes, as well as extensive reaforestation efforts, have completed the first phase of land regeneration. The first structure of the city, the Matrimandir, is hearing commissioned. It is a large spherical structure with a diameter of 36 m, which is to be surrounded by 12 gardens in an oval shaped park that contains a Banyan tree and the amphitheatre with the urn. The central park is to be surrounded by a lake.

1.3 Water Management Concept

The vision

A solitary banyan tree stands on a low barren plateau of red laterite, about 60 m above the sea which lies 5 km to the east. This tree is to become the centre of the city. Buildings will spiral outwards from it to form a spiral galaxy lay out as seen from above, surrounded by a belt of dense tropical forest. Vegetation will extend inwards again to the centre between the arms of the spiral, acting as the ‘green lungs’ of the city.

On the crown of the hill will be gardens, surrounded by a large lake. Within the gardens, an amphitheatre, a large spherical building (the Matrimandir), and the old banyan tree will mark the centre of the city, which is meant one day to accommodate 50,000 inhabitants.

The Presence

When Auroville was founded the hill was barren. Only a few palmyras have survived the centuries of deforestation. During the monsoon season, the sea was dyed terracotta-red with the eroded lateritic soil.

The rains have carved two canyons, about 20 m deep and 100 m wide in places, from the crown of the hill down to the sea. Only a very small portion of the rainwater that falls on the plateau remains in the ground, which allows for only two meagre harvests per year.

The water table of the first aquifer lies about 30 m below the compacted surface, where it can only be tapped in small quantities, with difficulty.

A layer of red laterite covers the entire hill and slopes gradually towards the sea. Under it lie strata of sandstone, clay and limestone, through which the groundwater flows out into the sea.

Adequate aquifers are found only at 100 m and 200-300 m depths.

City life could become possible on this site only if the area could be once more made fit for human habitation. The first step in this process was to protect the ground. The annual loss of soil and water could only be halted by creating "bunds", banks and dykes which slow down and divert runoff, and by terracing the land and stabilizing the canyon walls. These measures had to be undertaken throughout the entire city area.

The second step was to cover the surface with a layer of vegetation that would hold the soil together, as well as open it up so it could absorb the rainwater that fell on it.
The third step was to re-afforest the entire area. Pioneer trees and shrubs that could survive in extreme conditions had to be planted first, followed by others that could enable the subtropical forest to recover

Over the last thirty years, this process of topsoil regeneration, retention of rainwater to restore the groundwater, and modification of the micro-climate by providing shade, moisture and protection from wind and rain, has gradually brought the land back to life. The diversity of insect, bird, and animal species has consistently increased, further supporting the process of renewal of life.

The Conventional Solution

As there is no convenient surface water available in the area, drinking water for the city could be conventionally supplied by one or two central pumping stations, drawing groundwater from the aquifers at depths of 100 to 300 m. This is what is currently being done in neighbouring Pondicherry.

In this case, a large lake at the highest point of the city may not be the optimal solution, since it would have to be filled with groundwater drawn from great depths, which would require a lot of energy and be very expensive to operate and maintain.

To cover the other water needs of the city and its surrounding agricultural areas, a separate irrigation system, supplied by deep bore wells, would have to be set up.

The city receives an average annual rainfall of around 1200 mm, occurring within two rainy seasons, during which extreme downpours of up to 300 mm in 24 hours can occur . Runoff could be channeled into the canyons. Sewage could be collected in a conventional drainage system; purified in a conventional sewage plant; and be carried away through the canyons to the sea.

The Problem

The planned city of Auroville is situated at the downstream end of the aquifer, just before it runs into the sea. All the other users of the groundwater with access to the aquifers at 100 - 300 m depth have already been extracting their requirement. With powerful pumps, at subsidized electricity rates, agricultural users in the surrounding area, even the narrow strip directly along the coast, are removing groundwater at a very high rate to cultivate crops . In addition, a rapidly expanding industrial sector is making extravagant demands on the precious water supply.

The first signs of salt-water intrusion into the aquifers are already evident. South of the city, many square kilometres of coastal land have become infertile due to salination. Providing water for the needs of the city and its surrounding agricultural areas by desalinating sea-water is technically possible, but at present too costly to be affordable by the residents.

Salination of the groundwater could mean the end of the Auroville Project.

The Alternative

It does not seem to be a wise decision to build a city on a bare hill near the sea, on hard dry red earth, with a huge artificial lake at the highest point, where there exists a grave risk that the groundwater beneath it could become saline.

The chances of survival for this city seems slim when the lack of awareness in the surrounding area, over which it has little direct influence, is leading to pollution and over-extraction of the aquifer and the end / of its drinking water supply.

It has already been established that the food required by the city is to be provided from within its own greenbelt. If water is considered to be a part of the food of the city, this approach could be also applied to the provision of water.

Even in drought years, precipitation over the city area delivers more than ten times the amount of drinking water needed. But the rainy season lasts only for a few months. Collecting and storing all this rainwater would require huge tanks that would not only be expensive to construct, but would take up a lot of space.

However, the vision needs to be reflected upon, and the apparent disadvantages of the site need to be viewed as potentially useful. The upper layer of relatively impermeable laterite, together with the uppermost aquifer, form the entire plateau on which the city stands, and both slope gently towards the sea. Therefore, all of the water that percolates within the city area moves gradually above sea level towards the sea.

In this perspective, instead of seeing the terrain and soil as a disadvantage for the city, they can become blessings.

Because of the low permeability of the soil the groundwater is prevented from flowing downwards or towards the sea too quickly. Therefore in these circumstances in order to increase the groundwater resources, the best place to infiltrate surface water is at the highest point of the terrain, which happens to be at the centre of the city. From this point of view, the large lake that is envisioned at the city centre could be seen as an ideal element which could be integrated and be part of a technical solution for the problem of water management

Thus, in this perspective, the harvesting of rainwater that falls during the brief monsoon, and storing the harvested water in the uppermost aquifer, would appear to be a feasible option for securing the supply of freshwater to the city.

Rainwater runoff from the roofs can be collected in underground storages (cisterns) and used to substitute drinking water used for various household and gardening purposes.

The surface runoff from roads, paved surfaces and open areas can be collected and stored in reservoirs within the greenbelt at the boundaries of the city. After filtration, the stored rainwater could be gradually pumped up into the central lake, which means raising it no more than 20-30 m, by means of solar energy. thereafter the water would undergo further purification through natural means and would be made suitable for groundwater recharge. The purified water from the lake could be made to percolate into the groundwater table at that location. By constantly feeding the central lake with water from the reservoirs in the greenbelt, the water level of the lake would be kept constant, providing optimal conditions for high quality landscaping and park areas, along with desirable micro-climatic effects.

Sewage from the densely developed areas could be centrally purified in the greenbelt, and then re-used for irrigation purposes. If necessary, sewage, as well as secondary rainwater runoff, from the less densely developed areas can be purified in decentralized wastewater treatment systems such as root-zone treatment plants and reused on site for irrigation.

In this way, the geological and geographical "disadvantages" of the city's location could make a regime of rainwater conservation possible that would provide a plentiful supply of water for both drinking and irrigation, even if the underlying groundwater becomes eventually saline. The average rainfall is not only sufficient to support vigorous tropical vegetation, but would provide enough surplus to supply the surrounding areas.

However, this will be successful only if the residents of the city protect the first aquifer from contamination.

The upper strata of earth that lies beneath the city functions as a reservoir, and must therefore be protected. Drinking water can be obtained from wells that are located in the greenbelt that tap the groundwater before it flows beyond the city limits towards the sea.

The extreme degradation of life's basic elements through over-exploitation of this area's natural resources, has revealed the perfection of the original vision. This vision enables the residents of the city to live unaffected, even in the midst of a degraded environment, as long as they, themselves, avoid polluting the ground and the water, which together form the basis for their own survival.*

1.4 Contributions of The Mother to the Water Management for Auroville

After the water management concept for Auroville was presented by the author in January 1992 the literature on Auroville, The Mother and Sri Aurobindo was studied by several persons from Auroville and various quotations were discovered. In April 2000 a collection of quotations from The Mother were interpreted by the Matrimandir Coordination Group (MMCG) and published as an alternative concept. In 2002 this interpretation was reviewed by the chairman of the Governing Board who found it to be a misinterpretation of The Mother's words.

After the book "The Spirit of Auroville" was published by Huta D. Hindocha in 06/2002, the following quotation from The Mother dated 16.02.1968 from the book was presented during a Working Group meeting in Auroville on 30.06.2002:

> *"Your tiny house will be between the Mother's Shrine and the banyan tree. There will be 12 gardens with various kinds of Hibiscus and other flowers, plants, tall trees with marble seats underneath. Marble statues, marble fountains, small waterfalls, small pools with different coloured lilies and lotuses, small bridges, rockeries in Japanese style with varieties or cactus. There will be only one entrance. The pavement will be decorated with precious and semi-previous stones.* ***This area will be surrounded by a huge lake****. On one side of it, there will be tall trees – they mean Unity. On the other side of the lake there will be hillocks with fir and pine trees. The Mother's Shrine will be on an* ***island****."*

* This concept was developed by the author in December 1991 and presented for the first time to the Auroville public at Auroville's Centre for Scientific Research (CSR) on 10.01.1992.
In 1996 this concept was the basis of a rainwater management plan for a project near Berlin, in Teltow-Mühlendorf. The main elements of the concept have been realised in two phases in a zero-runoff settlement of 30 ha.
This concept has become the model for 4 other projects in Germany, one of which has a size of 30 ha and is almost completed is the project LandsbergerTor in Berlin-Marzahn. Another project, Schweriner Hof in Berlin-Hellersdorf is already completed.

During the same meeting a second quotation from The Mother to her disciple Satprem was mentioned dated December 31, 1969 in the compilation of “L’ORATOIRE DE LA MERE” that refers to the size of the lake:

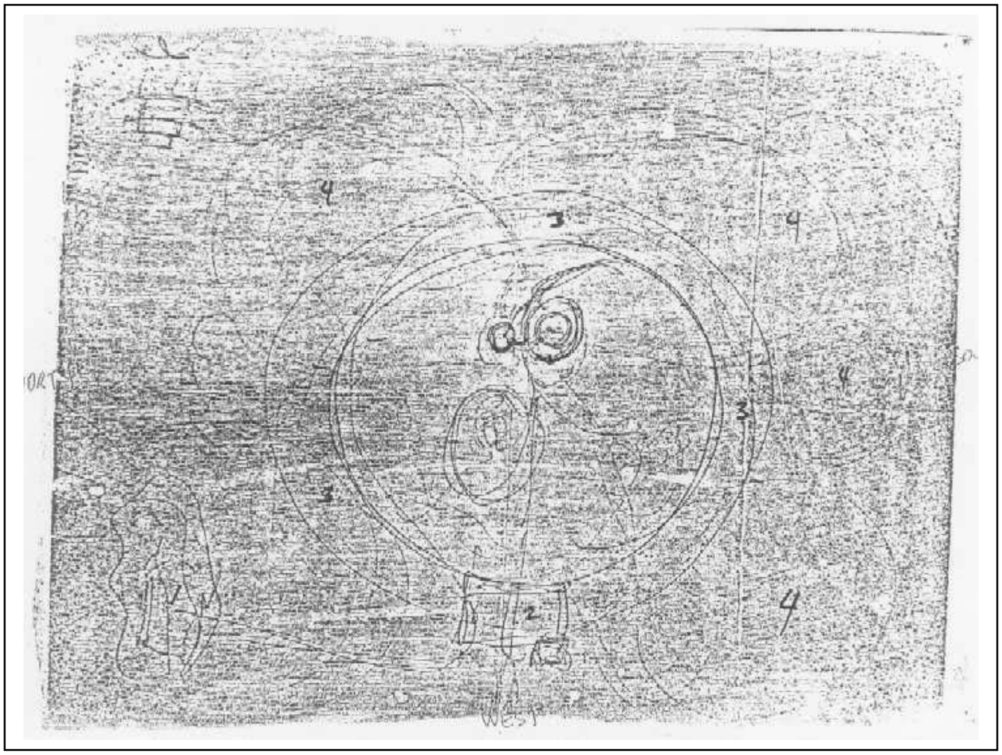
> “(After a silence) Roger’s idea is to have an island in the centre surrounded by water, running water which will be **used for the city’s whole water supply**; and when it has gone through the city, it will be routed to a mill and will be used to irrigate the environing agriculture.”

It was pointed out that she had not rejected this idea, and by stating it in her conversation she had expected to implement the idea.

The vision of the lake surrounding the Matrimandir (Mother’s Pavilion) dates back to 23.06. and 25.06.1965. Perhaps the strongest remark on the lake was made on March 28, 1970, in reply to a letter from Huta, where The Mother dictated to her son Andre M. the following:

> “It has been decided and will remain decided that the Matrimandir will be surrounded with water. However, water is not available just now and will be available only later, so it is decided to build the Matrimandir now and surround it with water only later, perhaps in a few years’ time.
> As regards the Matrimandir itself I have selected our plan, which agrees with the vision I had of the inside and have myself approved.
> Therefore there is no need to worry.
> The Matrimandir will be built now and water brought around it later.”

On 29.12.1998 Roger Anger presented a letter of an American Aurovillian named Narad dated 17.10.1977 to the author, showing a sketch from 25.06.1965 of the lake drawn by The Mother, giving clear directions for the location and layout of the lake.



1.5 Objectives

On September 10th 2001 Ingenieurbüro Kraft was awarded the contract to prepare a pre-feasibility study for the water supply, stormwater management and wastewater management for the City of Auroville.
With the Asia Urbs project of the European Commission, a partnership for urban development through partnerships with local governments, the following Terms of Reference for the study were agreed upon:

Pre-feasibility study for the stormwater management of the City of Auroville

a) Development of the technical solution and methods for the drainage, harvesting, re-use and groundwater recharge of the entire stormwater runoff from the city and its greenbelt.
b) Preparation of necessary hydraulic and structural calculations for the entire system, considering all potential town extensions.

c) Preparation of the preliminary design of the entire system, including primary and secondary collectors, pumping stations, lower storage basins, central lake, purification, and infiltration systems.
d) Estimation of the demand for maintenance of the entire system.
e) Preparation of a preliminary cost estimate for the construction of the entire system.
f) Proposing a preliminary phasing schedule for construction according to the priorities of urban development.

Pre-feasibility study for the water supply of the City of Auroville

a) Development of the technical solution and methods for the water supply of the entire city area using the first aquifer and the potential of the reuse of stormwater and wastewater to decentralize and to minimizing groundwater extraction.
b) Preparation of necessary hydraulic and structural calculations for the entire system, considering all potential town extensions.
c) Preparation of the preliminary design of the entire system, including wells, water works, primary and secondary distributors, pumping stations, and storage basins.
d) Estimation of the demand for maintenance of the entire system.
e) Preparation of a preliminary cost estimate for the construction of the entire system.
f) Proposing a preliminary phasing schedule for construction according to the priorities of urban development.

Pre-feasibility study for the wastewater management of the City of Auroville

a) Development of the technical solution and methods for the wastewater drainage, decentralized treatment and re-use with special emphasis on the principals of Ecological Sanitation (ECOSAN).
b) Preparation of necessary hydraulic and structural calculations for the entire system, considering all potential town extensions.
c) Preparation of the preliminary design of the entire system, including decentralized wastewater treatment plants, primary and secondary collectors, pumping stations, storage basins, and infiltration systems.
d) Estimation of the demand for maintenance of the entire system.
e) Preparation of a preliminary cost estimate for the construction of the entire system.
f) Proposing a preliminary phasing schedule for construction according to the priorities of urban development.

Additionally a survey would have to be conducted to evaluate the infiltration capacity of the soil in Auroville.
The soil survey was to be conducted between February 1st and 16^{th,} 2002. The pre-feasibility study was prepared between March 2002 and October 2002.

1.6 Survey and Transmissivity

Prior to the Pre-Feasibility-Study the Auroville Development Group had commissioned a survey of the entire area of the township to complete the Study of the Water Resources of Auroville to be executed from November 1999 to January 2002. The scope of this study was to analyze the potential of the Cuddalore Sandstone Aquifer in detail. Additionally, as part of that study it was proposed that a groundwater flow model of the first aquifer would be developed in order to evaluate the effect of groundwater recharge in this aquifer. Unfortunately, up to now this study has not been completed, nor has sufficient data on the lithology, transmissivity and groundwater flow patterns been made available. The survey has not been completed for the area of the Green Belt.

2 DESCRIPTION OF THE PLANNING AREA

2.1 Location

The Auroville Township is located about 5 km NW of Pondicherry. It covers an area of 19,6 km², and lies close to the sea coast. In the north, it is by the Kaliveli tank, and in the south by the Union Territory of Pondicherry. In the west, it is bordered by a topo "low" stretching in the NNE-SSW direction, and in the east by the Bay of Bengal. At the very centre of Auroville is the Matrimandir, which is the most important zone of the city. Auroville, as it currently appears, is shown in the Annex.

2.2 Physiography

The centre of the Auroville Township is located on high terrain at an altitude of about 52 m above mean sea level (MSL). Both steep and gentle slopes form the local terrain, the west and east having gradients of 0,6 and 1,1 percent, respectively. The high terrain runs in the NNE-SSW direction, as a parallel feature to the topo "low" in the west.

There is no perennial river system in the area. Along the high terrain, short flow courses of water in well-defined gullies are observed draining westward (topo "low"). Similarly, a few streams occur on the eastern slope of Auroville draining towards the sea. The natural shallow Kaliveli tank, located to the north of Auroville, covers an area of 76 sq. km and forms the main outlet for the gullies draining the northern and western parts of Auroville (see Annex).

2.3 Hydrometeorology

2.3.1 Rainfall

Rainfall data collected in Auroville for the period of **1972-83** is presented in Table 1. The rainfall data from Pondicherry is also available for the study.
The available data was compiled in the study by the Central Groundwater Board of Hyderabad “Hydrological Conditions in Auroville” (1984).

The annual weather cycle can be divided into four seasons:

a) Winter (January – February)
The average winter rainfall accounts for about one percent of the annual rainfall, having a standard deviation of 8 mm and a coefficient of variation of 81 percent.

b) Hot Weather Period (March – May)
The average rainfall received during this period is 45 mm, which amounts to 3 percent of the annual rainfall. The amount of rainfall has a standard deviation of 44 mm and a coefficient of variation of 97 percent.

c) Southwest Monsoon (June – September)
The south-west monsoon advances into Auroville and its adjoining areas by the end of May. During this period, the average rainfall over Auroville is 415 mm, which accounts for nearly 34 percent of the annual rainfall, with a standard deviation of 108 mm and a coefficient of variation of 26 percent.

d) Northeast Monsoon (October – December)
The average rainfall over Auroville during this season amounts to 759 mm, which accounts for 63 percent of the annual rainfall. It has a standard deviation of 280 mm and a coefficient of variation of 37 percent.

Table 2.1: Rainfall Statistics for Auroville for the Period 1972 – 1983

Details		Winter Period	Hot Weather	SW Monsoon	NE Monsoon	Annual
Mean rainfall	mm	10	45	415	759	1227
Seasonal rainfall as %-age of annual rainfall	%	1	3	34	62	---
Standard deviation rainfall	mm	8	44	108	280	342
Co-efficient of variation	%	81	97	26	37	28

The monthly rainfall data of Pondicherry from the period 1995 –1998, as well as the annual rainfall data from the period 1995 –1998, are shown in Annex 1.

2.3.2 Temperature

The maximum and minimum temperatures recorded in Auroville were 43,8°C in May, 1976, and 14,9°C in February, 1974, respectively. During the winter, the mean monthly temperatures are moderate, varying from 23,5°C to 25,4°C. But from March to May, the mean monthly temperatures range from 26,9°C to 31,1°C. During the south-west monsoon period, the temperatures remain high, but decline towards the end of the south-west monsoon (31,5°C to 28,5°C). During the north-east monsoon, the temperatures fall further (27,6°C to 23,8°C).

Table 2.2: The Monthly Maximum, Minimum and Mean Temperatures of Auroville between 1972 –1980

Details		Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Max. Temp.	°C	28.4	30.0	31.3	33.3	35.7	36.6	35.5	34.2	33.1	31.2	29.3	28.2
Min. Temp.	°C	18.8	19.9	22.1	25.3	26.3	26.3	25.5	24.2	23.8	23.7	21.7	19.6
Mean Temp.	°C	23.5	24.9	26.9	29.3	31.1	31.4	30.4	29.5	28.5	27.6	25.6	23.8

2.3.3 Evaporation

Evaporation data from a class “A“ pan in Cuddalore is considered as representative for the Auroville region. The data covers a 3-year period from 1981-1983. Evaporation was recorded for the four different weather periods as follows:

Winter	237 mm
Hot weather	498 mm
SW monsoon	694 mm
NE monsoon	301 mm

The mean annual evapo-transpiration at Cuddalore was recorded at 1,706 mm.

2.3.4 Potential Evapo-transpiration (Thornthwaite’s Method)

The mean monthly temperatures at the Auroville station were used for computing monthly potential evapo-transpiration. The figures are given in Tables 3 and 4. During the winter period, the average potential evapo-transpiration is 97 mm, while during the hot weather period, it increases to between 142 and 183 mm. During the south-west monsoon, it decreases again from about 145 to 94 mm.

2.3.5 Relative Humidity

Monthly relative humidity data in Auroville, taken at 08.00 hrs and 18.00 hrs for the period 1972-1981, is given in Table 5. Mean monthly relative humidity at 08.00 hrs and 18.00 hrs is depicted in Plate VI. The average relative humidity during the winter period is 81 % at 08.00 hrs, and 71 % at 18.00 hrs. During the hot weather period, it ranges from 81 % to 73 % at 08.00 hrs, and 75 % at 18.00 hrs. During the south-west monsoon, it ranges from 66 % to 81 % at 08.00 hrs, and from 64 % to 79 % at 18.00 hrs. During the north-east monsoon, the relative humidity is 87 % at 08.00 hrs and 80 % at 18.00 hrs.

Table 2.3: Monthly Potential Evapotranspiration (mm) at Cuddalore

Year	Jan.	Feb.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
1981	NA	NA	NA	158.1	197.7	392.1	209.9	184.4	129.1	102.4	86.7	89.2
1982	128.9	133.7	183.1	191.7	232.0	118.9	160.4	195.0	165.5	145.2	63.8	89.2
1983	99.7	111.2	116.4	109.5	156.6	212.0	140.1	98.7	74.4	106.0	108.3	113.2
Mean	114.3	122.5	149.8	153.1	195.4	211.0	176.1	159.4	123.0	117.9	86.3	97.2

Table 2.4: Mean Monthly Potential Evapotranspiration at Auroville Station

Details		Jan.	Feb.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.
Mean Temperature	°C	23.5	24.9	26.9	29.3	31.1	31.4	30.4	29.5	28.5	27.6	25.6
Potential Evapo-transpiration	mm	92	101	142	162	183	183	182	170	156	145	122

Table 2.5: Mean monthly relative humidity at 08.00 hours and 18.00 hours

Details	Jan.	Feb.	March	April	May	June	July	Aug.	Sep.	Oct.	Nov.
Rel. Humidity % at 08.	80	82	81	78	73	66	74	75	81	86	88
Rel. Humidity % at 18.	70	71	71	77	77	64	67	70	79	83	80

2.4 Geology

Throughout most of the Auroville region, sedimentary formations (laterite and sandstones) of Pleistocene and Mio - Pliocene are exposed. Sedimentary formations (clays and limestones) of the Palaeocene period are exposed in the most western part of the study area. Based on a study of aerial photos (scale 1:20.000), Mio-Pliocene and Palaeocene formations are demarcated. Exploration by the Oil and Natural Gas Commission (ONGC) and the CGWB in this region have brought to light the presence of sedimentary formations (claystones, limestones, and sandstones) from the Cretaceous age and crystalline rocks (charnockite) from the Archaean age, at depths below the Mio-Pliocene formations.
The detailed geological succession of formations is presented in Table 6, while a brief description of various formations is presented in the following paragraphs:

Table 2.6: Geological Succession in Auroville

ERA	Period	Formation	Lithology
Quarternary Tertiary	Recent Mio-Pliocene	Alluvium Laterite Cuddalore formation	Sands, clays, silts, kankar and gravels, laterite Sandstone, pebbly and gravely and coarse-grained with minor clays and slitstones and thin seams of lignite
		Unconformity	
Tertiary	Palaeocene	Menaveli formation	Yellow and yellowish brown, grey calcareous siltstone and claystone and shale with thin bands of limestone
Tertiary	Palaeocene	Kadaperikuppam formation	Yellowish white to dirty white, sandy, hard fossiliferous limestone, calcareous sandstone and clays
		Unconformity	
Mesozoic	Upper Cretaceous	Turuvai limestone	Highly fossiliferous limestone, conglomeratic at places, calcareous sandstones and clays
Mesozoic	Upper Cretaceous	Ottai Claystone	Greyish to greyish-green claystone with thin bands of sandy limestone and fine-grained calcareous sandstone

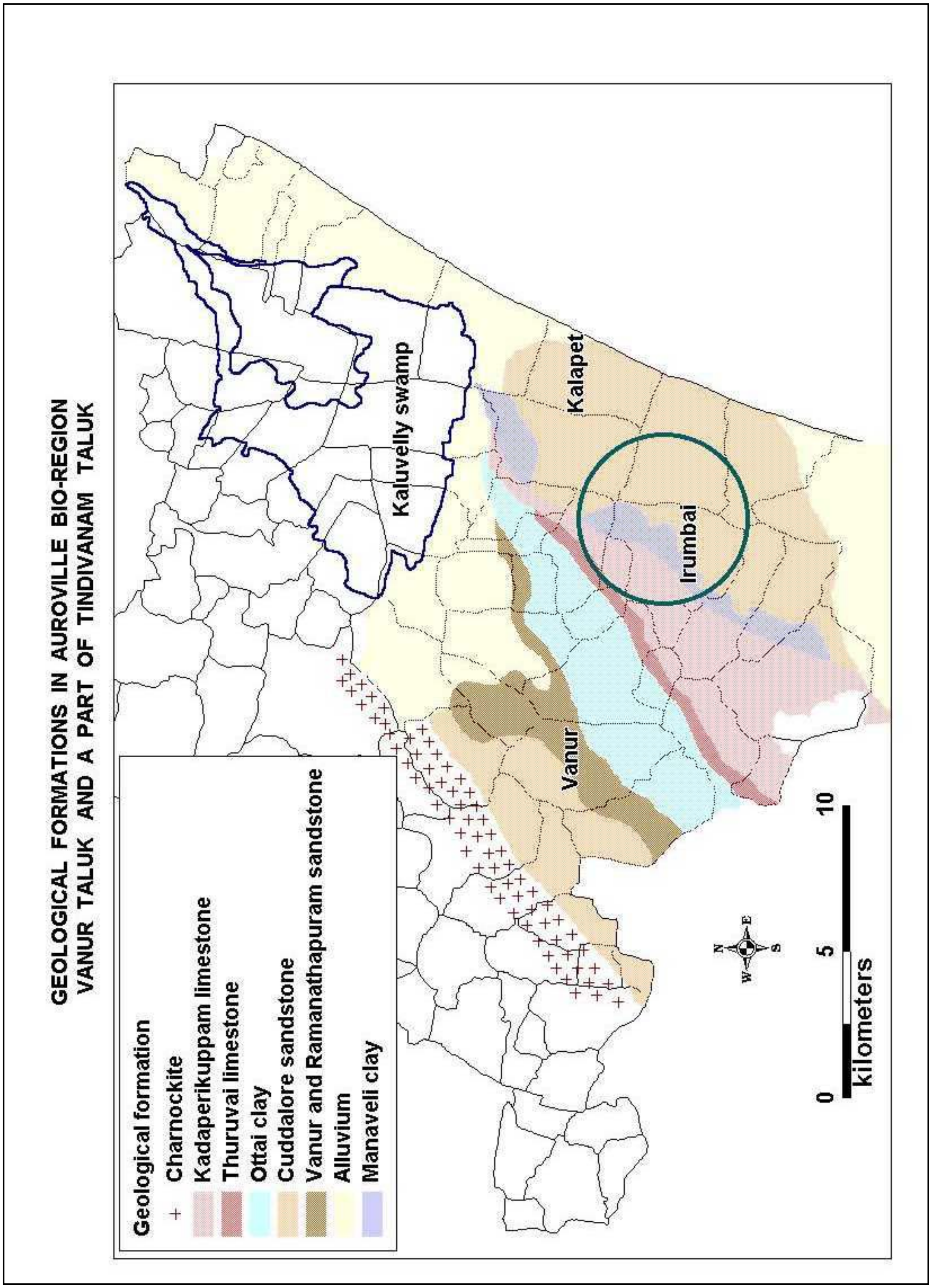
ERA	Period	Formation	Lithology
Mesozoic	Upper Cretaceous	Vanur Sandstone	Quartzose sandstones, hard, coarse-grained, occasionally felspathic, minor clays
Mesozoic	Lower Cretaceous	Ramanathapuram formation (unexposed)	Black carbonaceous, silty clays and fine to medium grained sands with bands of lignite and sandstones, medium to coarse-grained
Unconformity			
Archaean		Eastern Ghat Complex	Charnockite and biotite hornblende gneiss

The area chiefly consists of lateritic soils from the Quaternary age, followed by Mio-Pliocene, Palaeocene and Cretaceous. The exploratory drilling carried out down to a maximum depth of 450 m bgl in this area has brought to light the occurrence of a crystalline basement at a depth of 448 m bgl. The maximum thickness of the cretaceous formations overlying the crystalline basement is around 370 m. The tertiaries overlying the cretaceous sediments record a maximum thickness of about 100 m in the eastern part of Auroville. The maximum thickness of the laterite (Quaternary) observed is 7,5 m. The sediments of the Tertiary and Mesozoic eras are of much importance with regard to groundwater development.

2.5 Groundwater

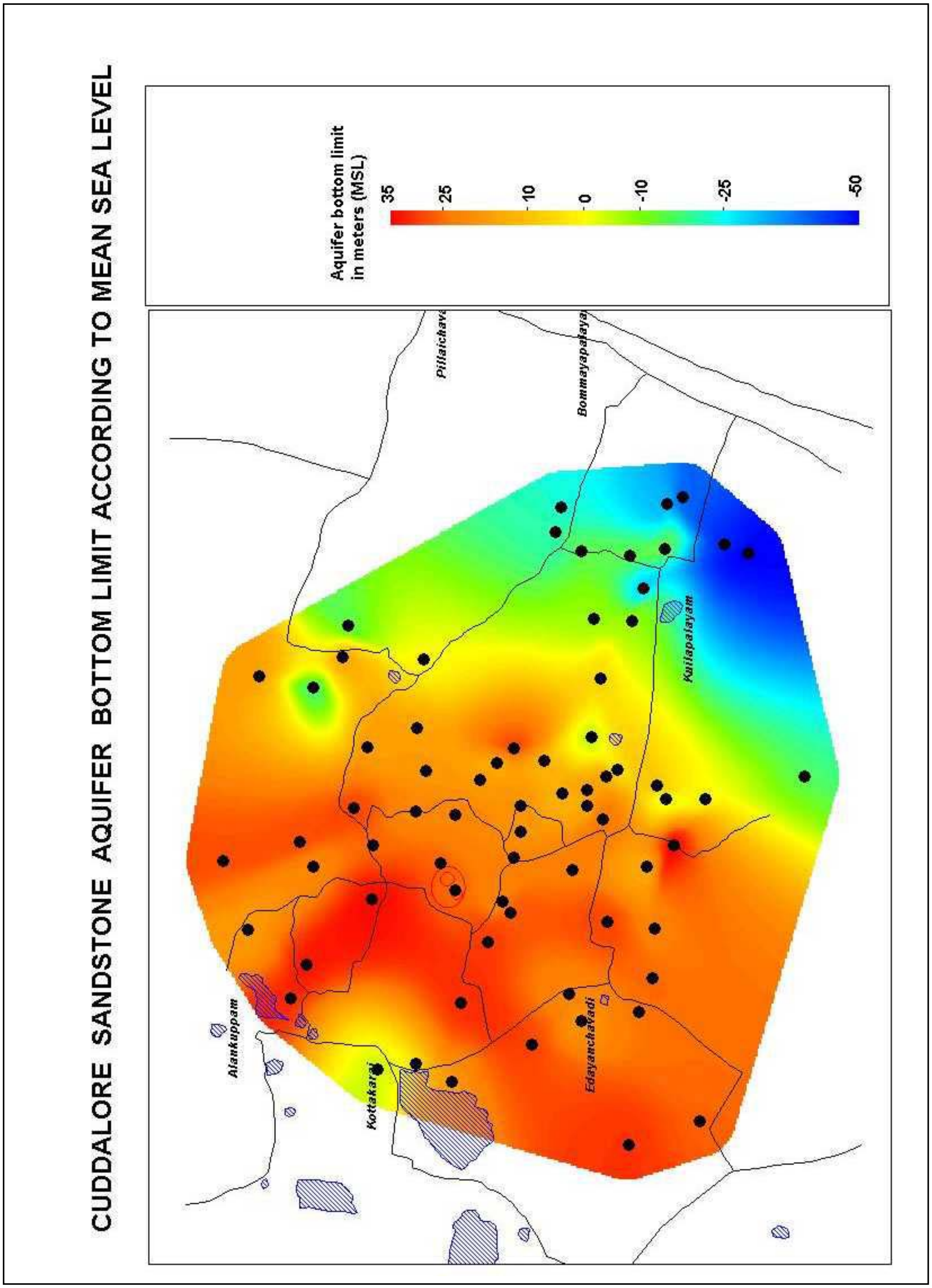
The geo-electrical soundings carried out at 11 sites, as well as the borehole logging of the available wells, have brought out interesting details with respect to sub-surface hydro-geological conditions. The top lateritic soil was reflected by resistivities of the order of 300 Ohm m. At most of the sites, lateritic soils are underlain by a moderately resistive (16 to 60 Ohm m) layer of thickness of 15 to 70 m, which is expected to represent the potential phrocatic aquifer. There is another thick and moderately resistive (10 to 23 Ohm m) geo-electric layer occurring at deeper levels below the predominating clay layer. Whenever the resistivity of the above layer is more than 15 ohm. m, the predominance of sandstones with productive granular zones could be seen at four sites. Further, the surveys indicated that the depth to the basement in the Auroville area is likely to be around 350 m to the west, and 550 m towards the coast with a south-easterly slope. The surveys also indicated that the chemical

Figure 2.1: Geological Formations in Auroville Bio-Region



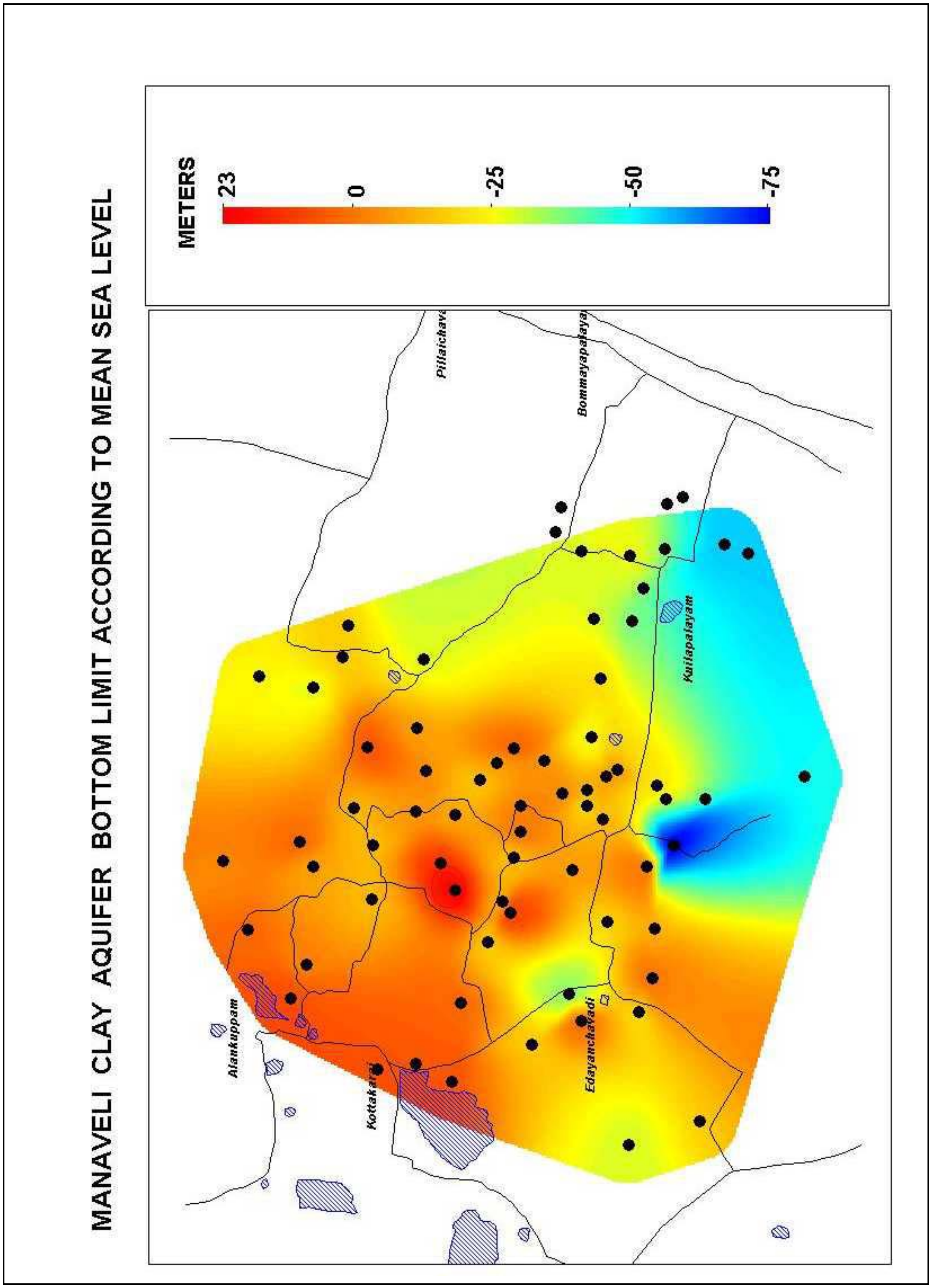
(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 2.2: Cuddalore Sandstone Aquifer Bottom Limit According to Mean Sea Level



(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 2.3: Manaveli Clay Aquifer Bottom Limit According to Mean Sea Level



(Source: Harvest, Interim Report on Auroville's Water Conditions)

quality of groundwater is good and suitable both for irrigation and drinking purposes throughout Auroville, except for regions to the east very close to the sea.

Groundwater occurs in the lateritic soil under phreatic conditions, as well as in the older formations of the Mio-Pliocene to the Cretaceous periods. This water can be tapped under the phreatic conditions with the depth to water table varying from less than a meter to 5,20 m bgl. The range of seasonal fluctuation in the water table is from 2,4 to 12,4 m.

Due to the existing shallow depths of the open wells, some of them go dry during the summer period. The yield from these wells ranges from 1,3 to 12,6 lps. The chemical quality of groundwater from the phreatic aquifers is generally good for both domestic and irrigation purposes.

Tubewells of 29,0 to 99,2 m depth, tapping the shallow confined aquifers of the Mio-Pliocene and Palaeocene ages, record a piozemetric surface of altitudes ranging between 3,7 and 45,0 m. These tubewells have yields varying from 0,17 to 7,8 lps. The permeability of the aquifers ranges between 1,8 and 4,2 m/day. The specific capacity values of tubewells vary between 0,085 and 0,29 lps/m of drawdown.

The preliminary assessment of groundwater resources in the study area revealed that the annual recharge to the water table aquifer, and the prevailing draft from it, are of the order of 3,48 M m^3 (average net value by both of the methods) and 0,30 M m^3 respectively. These values indicate high potential for further groundwater development from the water table aquifer with sound planning based on suitable large diameter open wells.

The total groundwater outflow from the Cuddalore and Kadaperikuppam aquifers (36,14 sq. km) was estimated to be 2,29 M m^3 annually, whereas the prevailing groundwater draft from the same aquifers is about 1,13 M m^3 annually within the Auroville region. Thus, the net reserves available work out to be 0,81 M m^3 in the Auroville area, which shows good promise for further groundwater development by dug wells or borewells, as well as shallow and deep tubewells.

The groundwater potential of the cretaceous sediments (i.e. Ottai claystones and Vanur-Ramanathapuram sandstones) in the study area could not be obtained as the available data with respect to these aquifers is insufficient. The present annual groundwater draft from this aquifer system, based on the regional hydro - geological data, shows that the groundwater outflow from this aquifer system is about 1,20 M M^3 annually. Thus, a quantity of about 1 M m^3 appears to be available for development through tubewells at the rate of 12,5 lps (45 m^3/hr) for 6 hours of pumping a day, and for 240 days. Detailed hydro - geological surveys and groundwater exploration are necessary before any large-scale groundwater exploitation programme can be planned with respect to these aquifers.

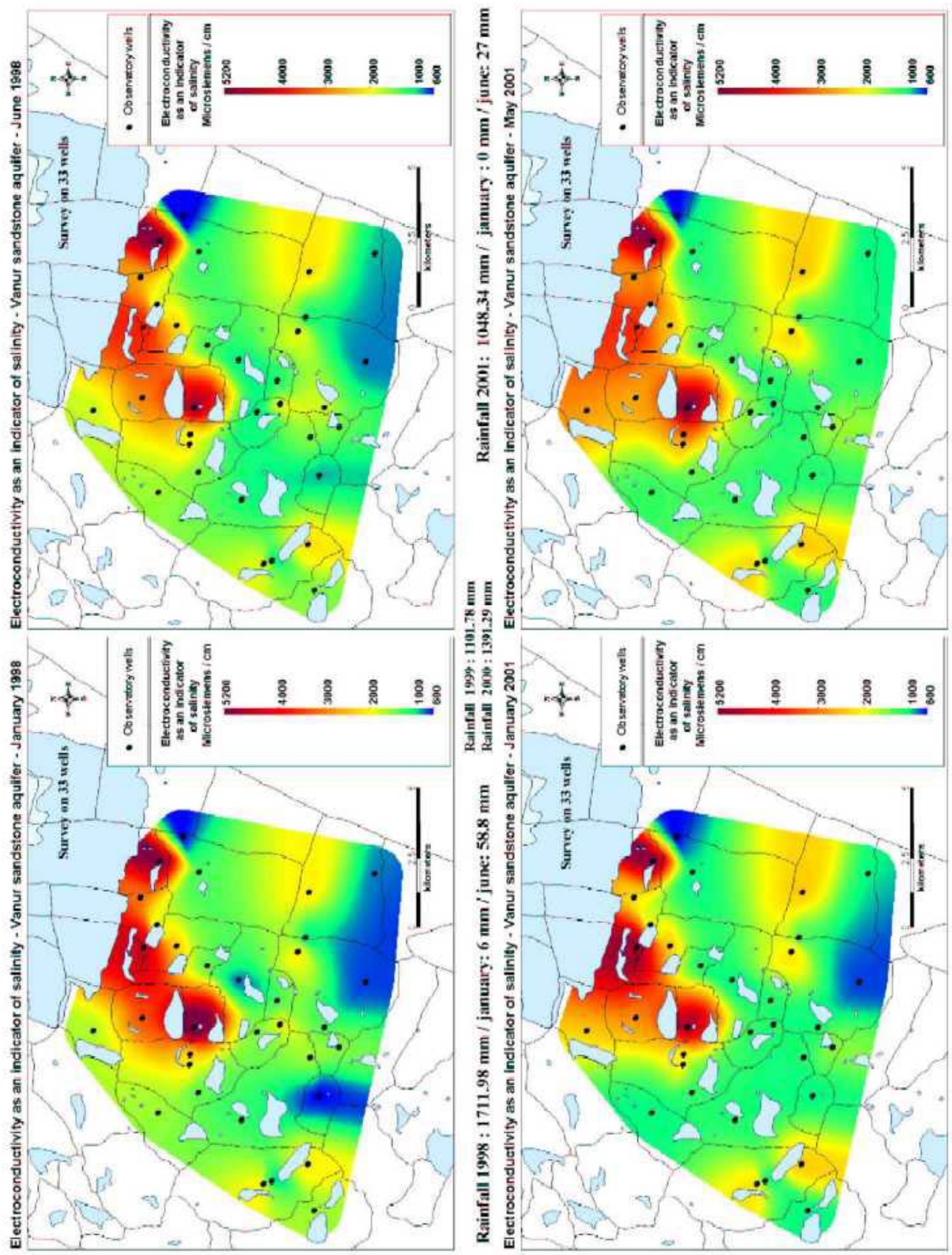
The chemical quality of the groundwater in the Cuddalore-Kadaperikuppam aquifer system is very good, with an electrical conductivity ranging from 92 to 475 microsiemen/cm at 25°C. The pH values range from 7,65 to 8,35. The groundwater is of the calcium-bicarbonate type. Chloride content varies from 7,0 to 28,5 ppm. The percent sodium and the Sodium Adsorption Ratio (SAR) values are also low, varying from 12 to 45, and from 0,3 to 0,76, respectively.

The Kaliveli tank, located to the north of Auroville, represents a natural feature caused by the structural disturbance (i.e. fault) extending in the NNE-SSW direction. The major portion of the tank is underlain by cretaceous sediments with a south-easterly dip. These sediments are encountered at certain depths in the Auroville area. The recharge conditions in the Kaliveli tank area, the hydraulic continuity, and the groundwater flow directions within these sediments, have to be studied in detail to understand the effect of the Kaliveli tank on the aquifer system in the Auroville area.
(Source:“Hydro geological Conditions in Auroville, Central Groundwater Board, Hyderabad, May 1984“)

2.6 Salt Water Intrusion into the Groundwater

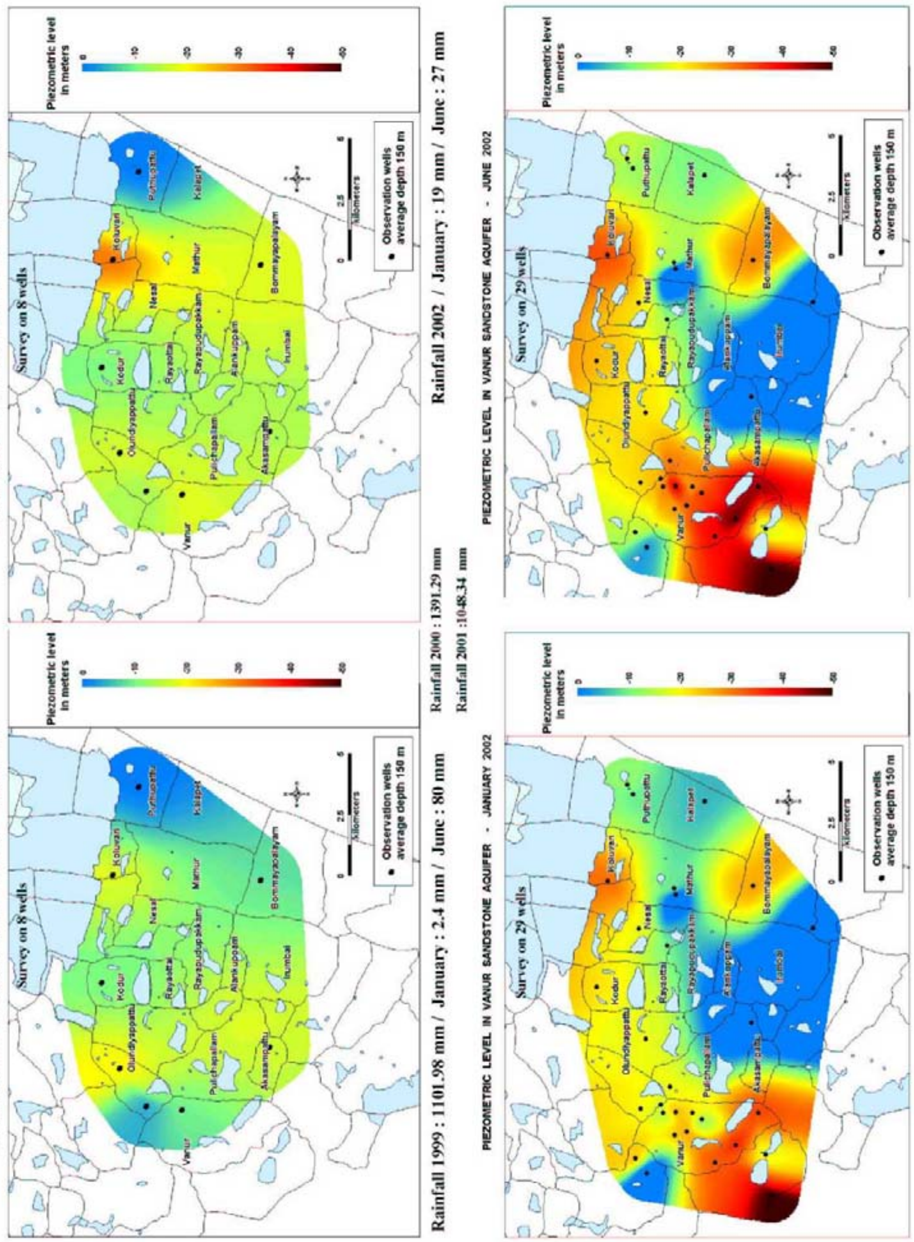
Since 1994, an increasing tendency towards salt water intrusion into the groundwater has been observed in the wells in the surrounding areas of Auroville. An increasing rise in salt content has been observed in some wells within the area of interest.
It is assumed that the uncontrolled and excessive groundwater withdrawal for irrigation purposes in the surrounding areas is the fundamental cause of saltwater intrusion.
The supply of electricity for irrigation is subsidised, and the boring of wells along the coastal strip is standard practice inspite of prohibitions.

Figure 2.4: Progress of Salinisation in Vanur Sandstone Aquifer in 1998



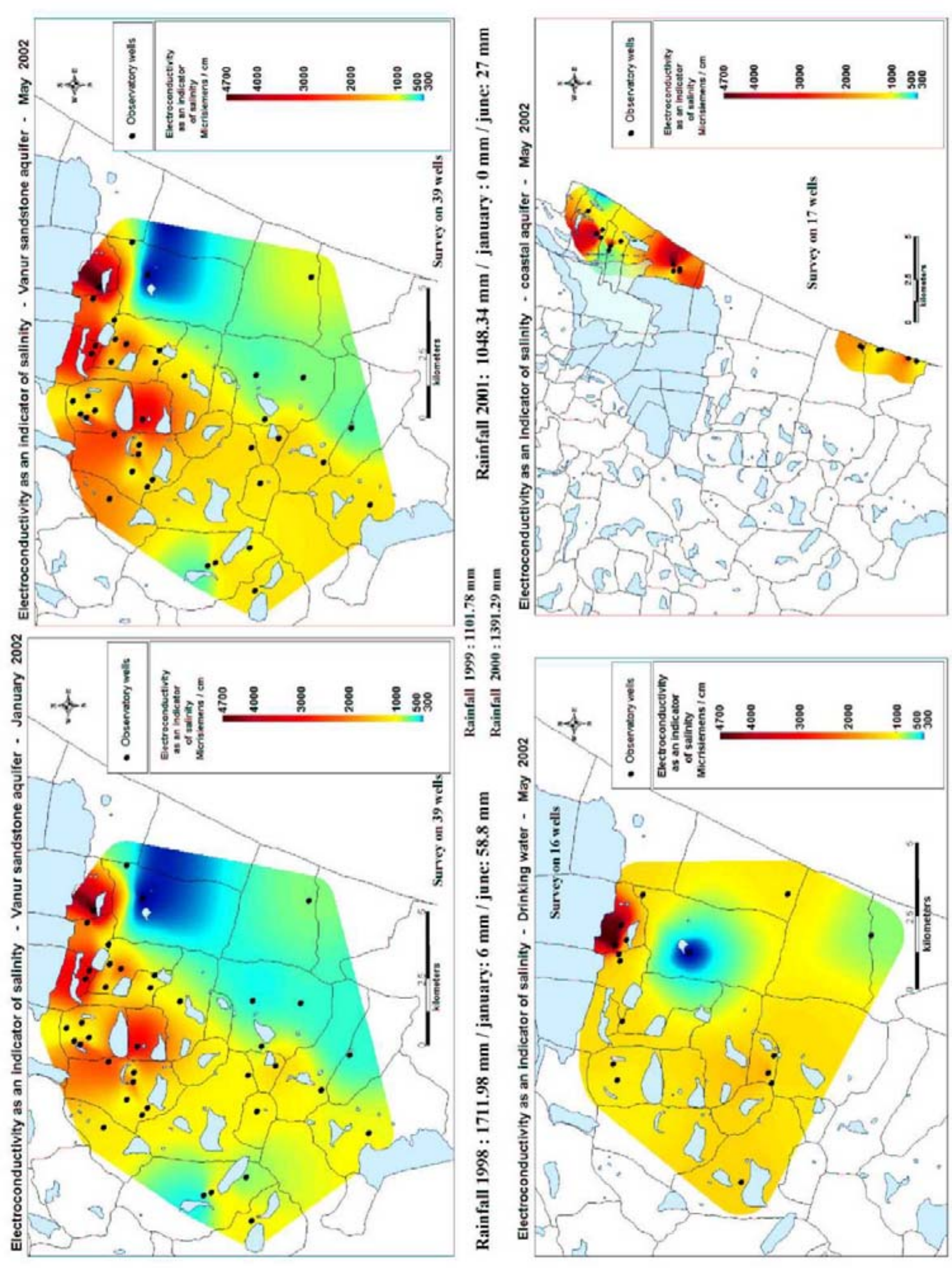
(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 2.5: Progress of Salinisation in Vanur Sandstone Aquifer in 1999



(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 2.6: Progress of Salinisation in Vanur Sandstone Aquifer in 2002



(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 2.7: Electric Conductivity of Groundwater in Cuddalore and Vanur Aquifers in Auroville, November to March 2002

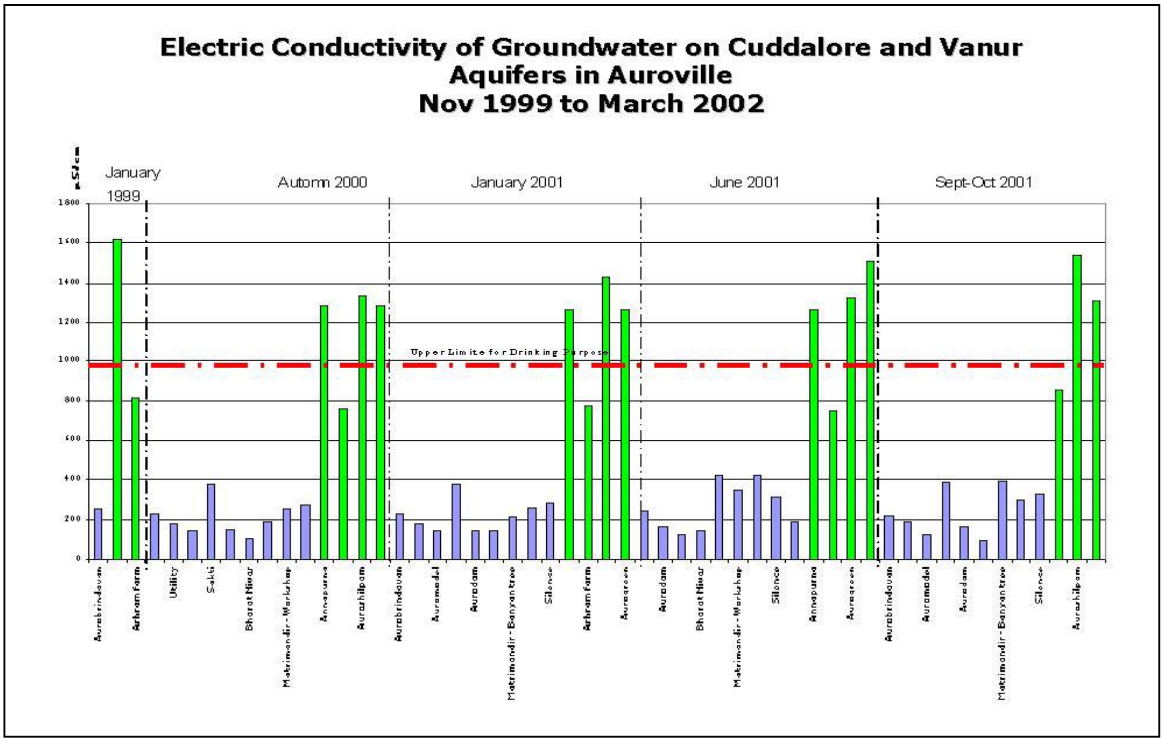
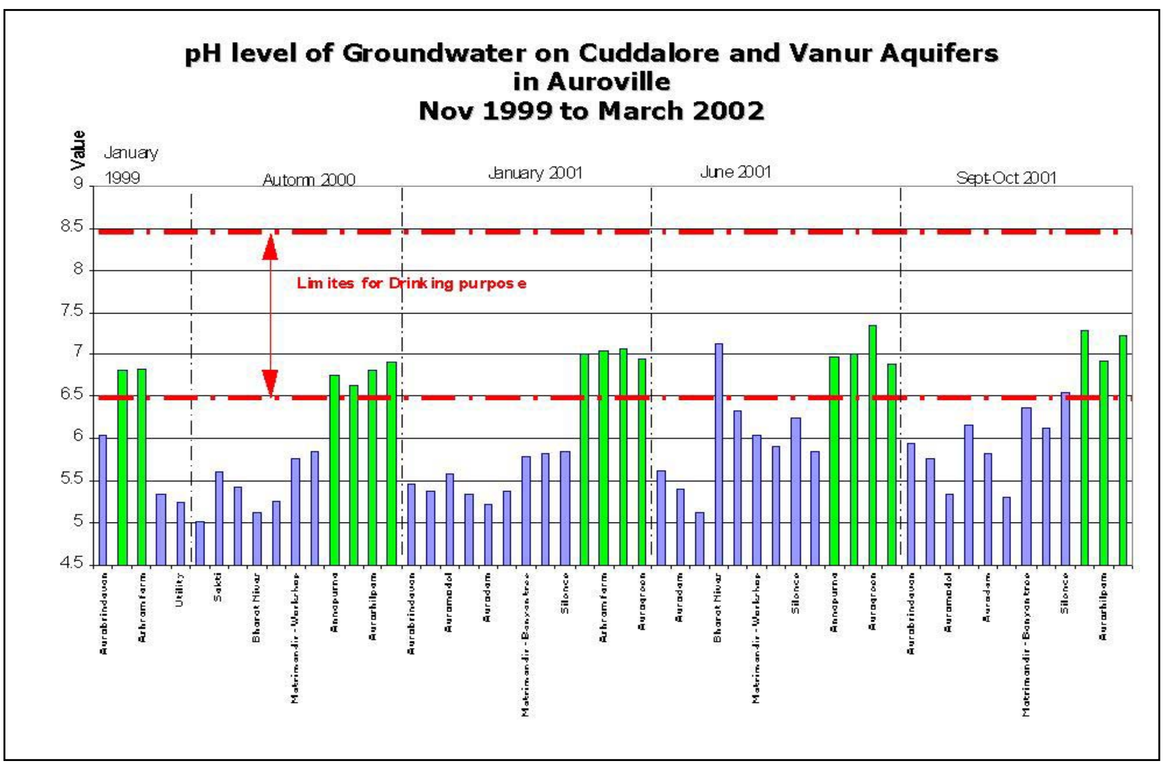


Figure 2.8: pH level of Groundwater in Cuddalore and Vanur Aquifers in Auroville, November to March 2002



(Source: Harvest, Interim Report on Auroville's Water Conditions)

3 WATER MANAGEMENT CONCEPT

3.1 Background

The Water Management Concept was presented by the author in January 1992 and was discussed in the community after that. In January 1999 the Development Group of Auroville requested the author to finalize the Water Management Concept on the basis of the Master Plan and to provide an independent Concept Report. The Water Management Concept Report was presented in September 1999 and served as the basis for further intensive discussions on the Water Management of the city.

The Auroville administration and the chief town planner Roger Anger integrated this concept into the Master Plan and decided to proceed with the planning process and subsequently submitted the preparation of a Pre-Feasibility Study to the EU /Asia Urbs Programme for funding.

The Pre-Feasibility-Study is based on the Water Management Concept Report of September 1999.

3.2 Water Resources in Auroville

3.2.1 Safe Water Yield from Precipitation

The precipitation over the urban area is, on the average, enough to cover the drinking water demand for the city (145 %). Only in a dry year it is possible that the water demand may not be completely satisfied (-22%).

Therefore for using the available rainwater for drinking water purposes, it is necessary that all precipitation which exceeds the long term average be completely used for recharging the groundwater.

3.2.2 Safe Water Yield from Sewage

To cover the water demand for drinking and irrigation purposes, it is proposed to feed back all of the urban sewage into the water cycle. The sewage will be treated to the extent that it can be re-used for irrigation (3,28 M m³/a.).

3.2.3 Water Balance

With a long-term average, the surplus of rainwater available for groundwater recharge amounts to 0,64 M m³/a. In a year with above average precipitation, the surplus water amounts to 1,85 M m³/a. In a dry year, however, the water supply for the city falls short by up to 0,58 M m³/a.

3.2.4 Drinking Water Supply

The drinking water demand for 50,000 inhabitants amounts to 3,65 M m³/a.
The runoff from rooftops, with a total surface area of 1,23 km², amounts to between 0,77 - 2,10 M m³/a., or 1,44 M m³/a. on an average.

All of the runoff from rooftops is to be stored in cisterns, from where it is either directly used to substitute drinking water or conveyed to a central infiltration facility. The specific cistern volumes would need to be at least 800 l/m² roof area, and better yet, 1,200 l/m² roof area. The total volume of all the cisterns in the city then amounts to 0,984 M m³ - 1,476 M m³. Approximately 40 % of the drinking water demand could be met from water stored in the cisterns.

The remaining 60 % can be met with the surface runoff from the streets, open areas and green areas. This runoff amounts to 2,70 – 5,60 M m³/a., or 3,84 M m³/a on average. The surface runoff could be intercepted by water courses within the greenbelt and then delivered to the city centre for groundwater recharge. From the central groundwater recharge facility, the groundwater needs a flow time of about 1 to 5 years to reach the city and the greenbelt limits. From the recharged 1st aquifer, 60 % of the drinking water demand can be drawn from 30 – 50 m deep wells that would be distributed throughout the entire greenbelt.

3.2.5 Sewage Disposal

In decentralized facilities located in the upper rim of the greenbelt, urban sewage (2,74 M m³/a.) is to be biologically treated, purified, and made available for irrigation in the agricultural areas.

3.3 Required Water Management Facilities

3.3.1 Central Infiltration Facility

The surface runoff from within the city limits is to be completely used for groundwater recharge. The most appropriate location for the infiltration facility is the city centre, since from here the path of flow to the edge of the city is maximized. The garden around the Matrimandir is, from a hygienic / point of view, by far the most preferable location for the proposed groundwater recharging facilities.

The proposed groundwater recharging facilities consisting of infiltration trenches that would be located along the most important passages would total in length about 2,150 m. The maximum daily infiltration capacity would amount to approximately 74,000 m³/d. The required infiltration capacity depends on the allocated storage volume for the surface runoff in the greenbelt.

In an average year, the maximum infiltration capacity required during the NE monsoon amounts to approximately 20,000 m³/d, and during an above average NE monsoon, approximately 38,800 m³/d would be required.

3.3.2 Storage Volume in the GreenBelt

The surface runoff from the city and the greenbelt would have to be intercepted at the fringes of the city in water reservoirs and continually transported to the city centre for infiltration. The size of the water reservoirs determines the size of the required daily infiltration capacity, as well as the size of the treatment plant and the retention time in the central lake. The larger the water reservoirs, the smaller the remaining facilities can be.

With a storage volume of 1,033 M m³, a precipitation of up to 350 mm can be stored.

The minimum required infiltration capacity would then be 33,600 m³/d, and the retention time in the central lake would be 41 days. When the storage volume is 3,983 M m³, the infiltration capacity as well as the inflow into the central lake can be reduced to only 13,300 m³/d due to equalization of the flows. The inlet filters would need to be only 3,000 m², and the average retention time would be 104 days.

3.3.3 Central Lake at the Matrimandir

With a central lake having a surface area of 181,000 m², a maximum depth of 10 m, and a slope of the embankment of 1 : 3, the average depth would be 7,60 m, and the storage volume would be 1,376,000 m³. The minimum retention time, with a maximum inflow of 34,400 m³/d, would be approximately 40 days.

The surface area of the lake sealant is 185,800 m². The loss due to infiltration through the clay seal (vacuum condensed natural clay) amounts to approximately 15,450 m³/a. The loss due to evaporation amounts to approximately 54,300 m³/a. on average.

The retention time in the central lake should be several months since the lake will be used for the natural treatment of the polluted surface water that is harvested during the rains.

3.3.4 Filters

The polluted surface waters that are harvested during the rains and stored in the greenbelt are to undergo extensive treatment before they are conveyed to the central lake. For this purpose, large capacity slow sand filters (0,2 m/h) are planned. The retained surface runoff from all the storage facilities within the greenbelt is to be passed through inflow filters located before the water enters the central lake.

The water inlet to the lake is to be designed so that an optimal distribution of the inflowing water is achieved and no disruption to the flow can take place.

Water flows out of the lake when it overflows through structures that are located at various depths, and are positioned on the other and opposite side of the point of inflow of the lake in order to maximize the flow times. The outflow water from the lake is to be cleaned and algae and other filterable materials removed before they reach the infiltration system. will be achieved by means of an outflow filter, planned as a rapid filter (10 m/h).

3.3.5 Power Requirement for the Conveyance of Surface Runoff

From the water reservoirs in the greenbelt, the surface water is to be conveyed by means of pressure conduits to the inflow filter at the central lake.
With an average vertical rise of 25 m, and an annual output of 2,07 – 5,6 M m³/a., the power requirement amounts to 277,423 kWh/a. – 750,517 kWh/a.

3.4 Calculations

3.4.1 Basis for Calculations

Precipitation:*	Average	1,300 mm (1.296 mm)
	Minimum	700 mm (729 mm)
	Maximum	1,900 mm (1.898 mm)

Evaporation: (Class A pan Cuddalore 1981-1983)*

Winter Period:	237 mm
Summer Period:	498 mm
South-West Monsoon Period:	694 mm
North-East Monsoon Period:	301 mm
Average annual evapo-transpiration:	1,600 mm

Temperature:*	Maximum:	43,8°C in May 1976
	Minimum:	14,9 °C in February 1974
	Winter:	Average 23,5 – 25,4°C
	March - May:	Average 26,9 – 31,1°C
	SW Monsoon:	Average 31,5 – 38,5°C
	NE Monsoon:	Average 27,6 – 23,8°C

*Central Ground Water Board
Hydrological Conditions in Auroville, Hyderabad May 1984

Relative Humidity (in Auroville 1972-1981):*

	8.00 a.m.	6.00 p.m.
Winter:	81 %	71 %
SW Monsoon:	66 - 81 %	64 - 79 %
NE Monsoon:	87 %	80 %

City area

City	Ø 2,5 km	A_c = 4,9 km²
Green Belt	Ø 5,0 km	A_{GB} = 14,7 km²
Total area		A_{tot} = 19,6 km²

Impervious areas	50 %	= 2,45 km²
Rooftops	25 %	= 1,23 km²
Streets, sidewalks, and public squares	25 %	= 1,23 km²
Open areas	50 %	= 2,45 km²
Garden areas	10 %	= 0,25 km²
Agricultural areas	50 %	= 7,35 km²
Wooded areas	50 %	= 7,35 km²

Population: 50,000 inhabitants

3.4.2 Safe Water Yield from Precipitation

3.4.2.1 Precipitation

Table 3.1: Rainwater Yield

Area	Rainwater Yield		
	Average M m³	Minimum M m³	Maximum M m³
City	6,37	3,43	9,31
Green Belt	19,11	10,29	27,93
Total	**25,48**	**13,72**	**37,24**

*Central Ground Water Board

Hydrological Conditions in Auroville, Hyderabad May 1984

3.4.2.2 Runoff

Table 3.2: Runoff

	km²	%	M m³	M m³	M m³
Rooftops	1.23	90	1.44	0.77	2.10
Streets	1.23	80	1.28	0.69	1.87
Open areas	2.45	20	0.64	0.34	0.93
Wooded areas	7.35	10	0.96	0.52	1.40
Agricultural areas	7.35	10	0.96	0.52	1.40
Total Runoff	**19.61**		**5.28**	**2.84**	**7.70**

Sewage Flow

Day	50,000 P x 200 l/P x 0,9	=	9,000 m³/d	= 0,009 M m³
Month	9,000 m³ x 30 d/month	=	270,000 m³/month	= 0,270 M m³
Year	9,000 m³ x 365 d/a	=	3,285.000 m³/a	= 3,285 M m³

3.4.3 Water Demand

3.4.3.1 Drinking Water Demand

Day	50,000 P x 200l/P x d	=	10,000 m³/d = 0,010 M m³
Month	10,000 m³/d x 30 d	=	3,000.000 m³/month = 0,300 M m³
Year	10,000 m³/d x 365 d/a	=	3,650.000 m³/a= 3,650 M m³

3.4.3.2 Irrigation Demand

Potential Evapo-transpiration 1,600 mm/a.

The water demand from the vegetation in the parks and the greenbelt is equivalent to the precipitation minus the runoff.
No other additional irrigation is planned.

The water demand for gardens and agricultural areas is equivalent to the potential evapo-transpiration.

Table 3.3: Water Balance for Irrigation

Water Balance		Average	Minimum	Maximum
Evaporation	m	1,60	1,60	1,60
Precipitation	m	1,30	0,70	1,90
Runoff	m	0,26	0,14	0,38
Water Deficit	**m**	**0,56**	**1,04**	**0,08**
Irrigation Demand				
Gardens	M m³	0,14	0,26	0,02
Agricultural Areas	M m³	4,12	7,64	0,59
Total Irrigation Demand	**M m³**	**4,26**	**7,90**	**0,61**

3.4.4 Water Balance

Table 3.4: Water Balance

Water Demand	Average M m³/a	Minimum M m³/a	Maximum M m³/a
Drinking Water	3,65	3,65	3,65
Irrigation	4,26	0,61	7,90
Total Water Demand	**7,91**	**4,26**	**11,55**
Safe Water Yield	Average M m³/a	Minimum M m³/a	Maximum M m³/a
Rooftops	1,44	0,77	2,10
Surface	3,83	2,06	5,59
Sewage	3,28	3,28	3,28
Total Safe Yield	**8,55**	**6,11**	**10,97**
Water Balance	**+ 0,64**	**+ 1,85**	**- 0,58**

3.4.5 Drinking Water Supply

Table 3.5 Drinking Water Supply

Safe Water Yield	Minimum M m³/a	Average M m³/a	Maximum M m³/a
Drinking Water Demand	3,65	3,65	3,65
Rainwater from the Rooftops	0,77	1,44	2,10
Rainwater from the Streets	0,69	1,28	1,87
Rainwater from the Open Areas	0,34	0,64	0,93
Rainwater from the GreenBelt	1,04	1,92	2,80
Total Rainwater Runoff	**2,84**	**5,28**	**7,70**
Water Balance	**- 0,81**	**+ 0,19**	**+ 1,95**
	78 %	**105 %**	**153 %**

The precipitation distribution in the rainy season is as follows:

		Average	Minimum	Maximum
SW Monsoon	34 %	442 mm	238 mm	646 mm
NE Monsoon	62 %	806 mm	434 mm	1178 mm

With a maximum precipitation of 1,250 mm in 4 months, and a maximum monthly precipitation of 400 mm, the surface runoff is estimated as follows:

Table 3.6: Surface Runoff

Catchment Area	Area km²	Runoff Coefficient %	Area km²	Runoff max. month M m³	Runoff NE-Monsoon M m³
Rooftops	1.23	90	1.11	0.44	1.39
Streets	1.23	80	0.98	0.39	1.23
Open Areas	2.45	20	0.49	0.20	0.61
Wooded Areas	7.35	10	0.74	0.30	0.93
Agricultural Areas	7.35	10	0.74	0.30	0.93
Total	**19.61**	**20**	**4.06**	**1.63**	**5.09**

The stormwater runoff from the rooftops will be directly utilized.
The surplus runoff is to be stored and eventually infiltrated.

3.4.6 Dimensioning of Water Management Facilities

Infiltration Facilities

Specific Infiltration Capacity

Width / Depth B / H = 1,0 m / 1,5 m

Infiltration trench (Rigolen)

$Q_s = k_s \times F_s \times (h+ü) / h$

Q_s = 0,0001 m/s x 0,5 x 4,0 m² x (1,5 m + 1,5 m)/1,5 m = 0,0004 m³/s

Q_s = 0,4 l/s = 34,56 m³/d x m

Table 3.7 Infiltration Trenches

Length of Infiltration Trench			Infiltration Capacity		
	No.	m	m³/d	M m³/month	M m³/a
Oval road	U_1	1,035	35,769	1,073	13,056
Circular Path	U_2	452	15,621	0,047	5,702
Total		**1,487**	**51,390**	**1,120**	**18,757**
	L_1	105	3,629	0,109	1,325
	L_2	74	2,557	0,077	0,933
	L_3	60	2,074	0,062	0,757
	L_4	49	1,693	0,051	0,618
	L_5	43	1,486	0,045	0,542
	L_6	44	1,521	0,046	0,555
	L_7	52	1,797	0,054	0,656
	L_8	65	2,246	0,067	0,820
	L_9	169	5,841	0,175	2,132
Total		**661**	**22,844**	**0,685**	**8,338**
Total Length		**2,148**	**74,234**	**1,805**	**27,096**

Flow time for the Recharged Groundwater

The flow time for groundwater occurring in the centre of the city to reach the city limits has been computed in the following manner:

The distance to the sea is 5,000 m.

The maximum difference in elevation is 52 m.

The maximum slope is I_{max} = H/L = 52 m / 5,000 m = 0,0104 %.

Assuming that the groundwater at the edge of the city has already dropped to sea level, the following flow times result:

k_f = 5*10^{-3} m/s
L = 2,500 m
H = 40 m
I = H/L = 40 / 2,500 = 0,016
v = k_f x H/L = 5 x 10^{-3} m/s x 40 m / 2.500 = 8 x 10^{-5} m/s
T_f = 2,500 m / 8 x 10^{-5} m/s x 3,600 s x 24 h = 361 d

k_f = 10^{-3}
v = 1,6 x 10^{-5} m/s
T_f = 1,808 d = 4,95 a.

k_f = 10^{-4}
v = 1,6 x 10^{-6} m/s
T_f = 18.084 d = 49,55 a.

Minimum Required Storage Volume in the GreenBelt

Maximum Precipitation – NE Monsoon:	1,700 mm
Average Monthly Precipitation	350 mm

Required Maximum Capacity for Infiltration Trenches
Q_{inf} = A_{red} x 350 mm/month / 30 d = 2,95 x 0,35 / 30 = 34,400 m³/d

Required Storage Volume in the Greenbelt:
min V_{GB} = A_{red} x 350 mm = 2,95 x 0,35 x = 1,033 M m³
max V_{GB} = A_{red} x 1.350 mm = 2,95 x 1,35 x = 3,983 M m³

Required Storage Volume for the Central Lake around the Matrimandir

max. monthly Inflow Q_z = A_{red} x 350 mm = 2,95 x 0,35 = 1,03 M m³/month

Chosen parameters of the central lake at the Matrimandir

Surface area	A_{tot}	= 181,100 m²
Inner Embankment	A_i	= 36,377 m²

Outer Embankment	A_o	= 55,253 m²
Area of lake bottom	A_b	= 94,173 m²
Slope of Embankment	1:n	= 1:3
Maximum depth	t	= 10 m
Average depth	t_m	= 7,60 m
Volume	V_n	= 1,376,369 m³
Minimum retention time	t_r	= 40 days

Inflow Filter

Inflow	max Q_d = 34,400 m³/d
Filter velocity	v_F = 0,2 m/h = 4,8 m/d
Minimum filter size	A_F = 34.400 m³/d / 4,8 m/d = 7,167 m²
Chosen	72 m x 100 m = 7,200 m²
Alternative	Ø 96

Outflow Filter

Filter velocity	10 m/h = 240 m/d
Minimum filter size	A_F = 34.400 m³/d/240 = 143 m²
Chosen	10 m x 15 m

Minimization of the Storage Volume in the GreenBelt

Table 3.8: Storage Volume in the Green Belt

Year	P_{tot}	NE-Monsoon	Month	Infiltration Capacity		Storage Volume in the GreenBelt		Retention Time in Central Lake	Inlet Filter Size
	mm	mm		mm/mon.	m³/d	mm	M m³	d	m²
1995	888.1	640.0	5	128.0	12,600	130	0.324	109	2,625
1996	1,893,0	1,574,4	5	314.9	31,000	360	1.062	44	6,458
1997	1,936,7	1,707,4	5	341.5	33,600	350	1.033	41	7,000
1998	1,761,1	1,497,8	4	374.5	36,800	340	1.003	37	7,667
1998	1,761,1	1,528,4	5	305.7	30,100	460	1.357	46	6,271

Minimization of the Required Infiltration Capacity

Table 3.9 Infiltration Capacity

Year	Average Infiltration Capacity		Required Storage Volume in the GreenBelt		Retention Time in the Lake	Inlet Filter Size
	mm/month	m³/d	mm	M m³	d	m²
1995	74.0	7,300	309.1	0.9118	188	1,521
1996	157.8	15,500	785.7	2.3180	89	3,229
1997	161.4	15,900	925.2	2.7290	87	3,313
1998	146.8	14,400	910.8	2.6870	95	3,000
1995-98	**135.0**	**13,300**	**1,350.0**	**3.9830**	**104**	**2,771**

3.4.7 Infiltration and Evaporation Losses in the Lake

3.4.7.1 Infiltration Losses in the Central Lake

Losses through infiltration with a sealant made of 100 mm vacuum-condensed natural clay:

Embankments

Q_B = A_{tot} x k_f x H / L = 3,5 * 10^{-11} m/s x 91,630 m² x 5 m / 0,1 m

Q_B = 0,0002 m³/s = 13,85 m³/d = 5,057 m³/a.

Bottom of Lake

Q_S = 94,173 m² x 3,5 x 10^{-11} m/s x 10 m / 0,1 m = 0,0003 m³/s

Q_S = 28,47 m³/d = 10.394 m³/a

Q_{INF} = 15,450 m³/a.

Evaporation Losses

Precipitation:	P_{ave}	=	1,300 mm
Evaporation:	E_{ave}	=	1,600 mm
Deficit:	D	=	300 mm
Surface area:		=	181,000 m²
Evaporation loss:	Q_V	= A x D = 181,000 m² x 0,3 m	
	Q_V	=	54,300 m³/a.

Total Losses in Lake through Infiltration and Evaporation

Q_{EI} = 69,750 m³/a. ≅ 0,07 M m³/a.
This amount corresponds to about 1.3 % of the total average runoff (5,28 M m³) as well as 5 % of the storage volume, and a lowering of the water level of around 38,5 cm, or about 3,2 cm on an average each month throughout the year.

3.4.7.2 Estimation of the Losses in Storage in the GreenBelt

Storage Volume
min V_{GB} = 1,033 M m³
max V_{GB} = 3,983 M m³

Average Depth = 5,0 m

Surface Area
min A_{GB} = 206,600 m²
max A_{GB} = 796,600 m²
Evaporation Losses
max Q_E = 0,239 M m³
min Q_E = 0,062 M m³

Infiltration losses when sealing with 100 mm vacuum sealed natural clay

min Q_{INF} = 1,75 x 10^{-9} m/s x 206,600 m² x 3,600 s x 24 h = 31 m³/d = 11,315 m³/a.
max Q_{INF} = 1,75 x 10^{-9} m/s x 796,600 m² x 3,600 s x 24 h = 120 m³/d = 43,963 m³/a.

Total Losses to Storage in the GreenBelt
min Q_{EI} = 0,073 M m³
max Q_{EI} = 0,283 M m³

3.4.7.3 Total Storage Losses

min Q_{tEI} = 0,073 + 0,07 = 0,143 M m³/a.
= 3,7 % of average annual discharge (3,845 M m³/a.)
max Q_{tEI} = 0,283 + 0,07 = 0,353 M m³/a.
= 9,2 % of average annual discharge (3,845 M m³/a.)

3.4.8 Facilities for the Conveyance of Surface Water

Annual Output
Q_{fl} = 2,07 – 5,6 M m³/a.
Q_{max} = 33,600 m³/d = 388,9 l/s

The maximum capacity required for the pumps is calculated from the output at maximum water level in the lake and the corresponding vertical rise according to the following equation:

$$P_P = \frac{\rho\, g\, Q_{P\,\max}\, H}{1{,}000\ \eta} \quad (7-1)$$

ρ = 1,000 kg/m³
H = 25 m
Q_{pmax} = 400 l/s = 1,440 m³/h = 34,560 m³/d
H = 0,61

$$P_P = \frac{1\,kg/1 \cdot 9{,}81\ m/s^2 \cdot 400\ l/s \cdot 25\ m}{1{,}000 \cdot 0{,}61} = 160{,}83$$

According to manufacturers' instructions, an increase of about 20 % above the required pump capacity, P_P, is necessary as a safety measure for the estimation of the minimum required motor capacity, P_M:
P_M = 1,2 P_P = 192,99 kW

The Power Requirement
E_{min} = 2,07 M m³/a. / 0,00144 M m³/h x 192,99 kW = 277.423 kWh/a.
E_{max} = 5,6 M m³/a. / 0,00144 M m³/h x 192,99 kW = 750.517 kWh/a.

Figure 3.1: Water Management Concept (1992)

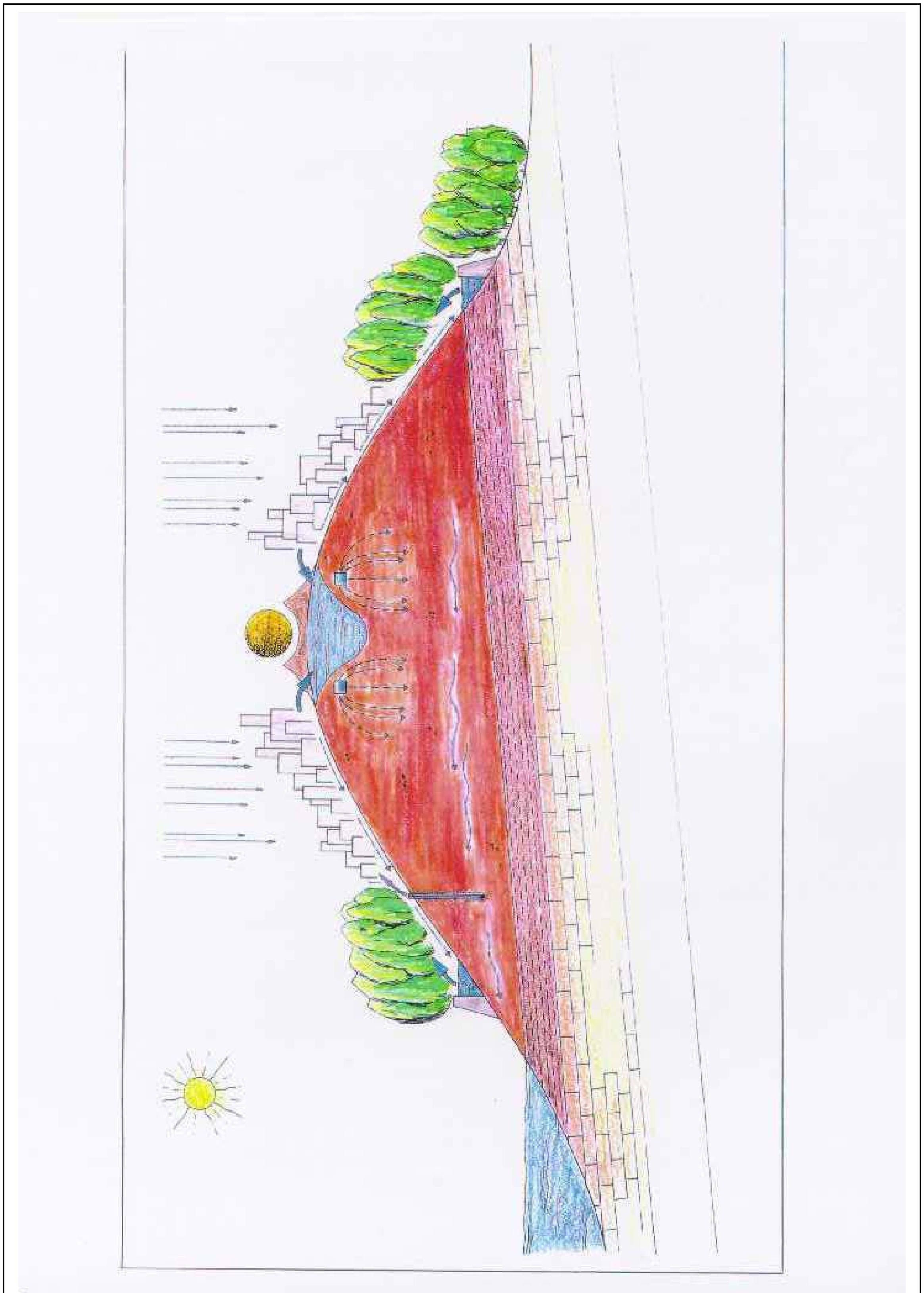


Figure 3.2: Matrimandir Lake (Visualisation by H. Loidl)

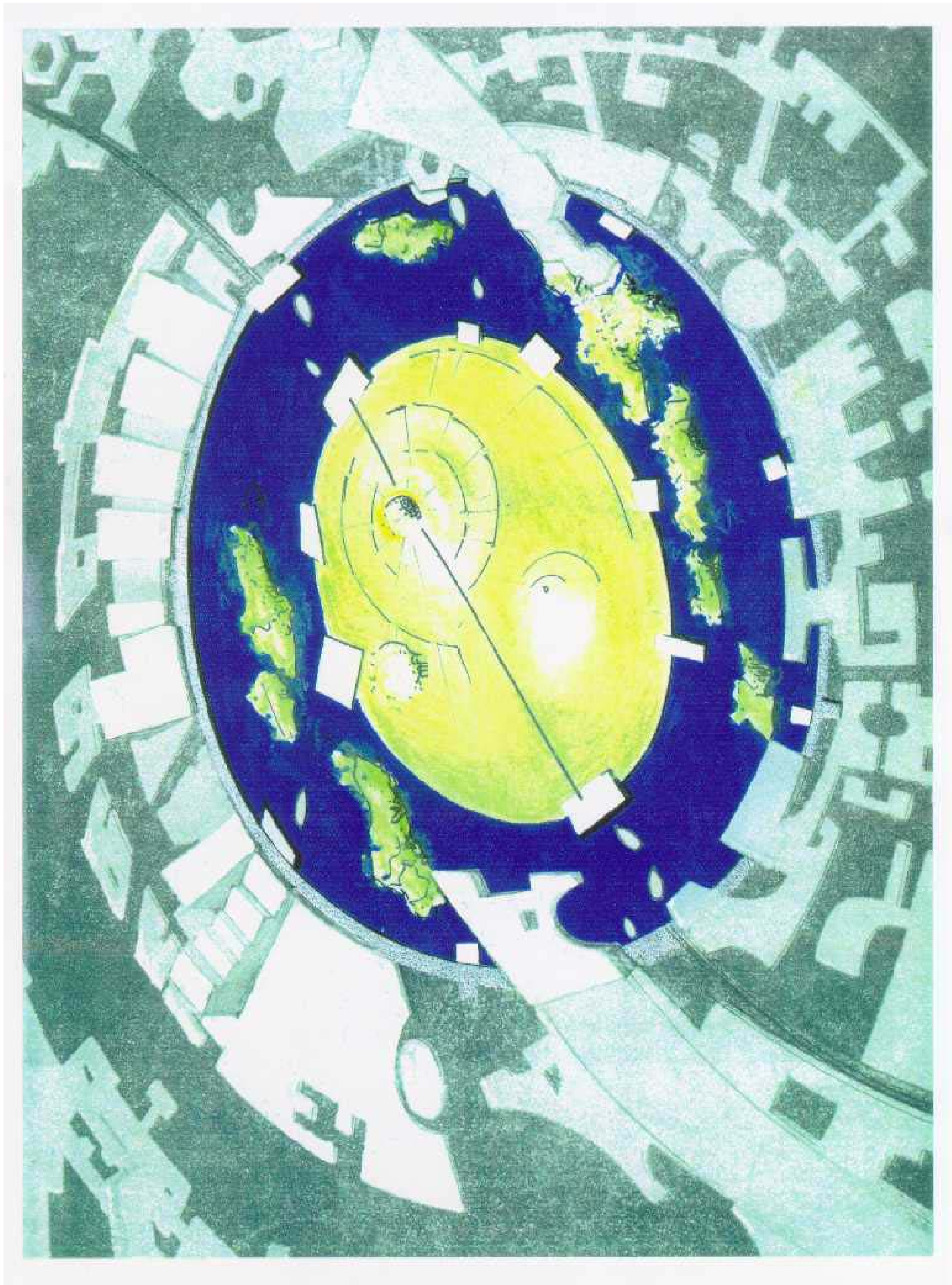
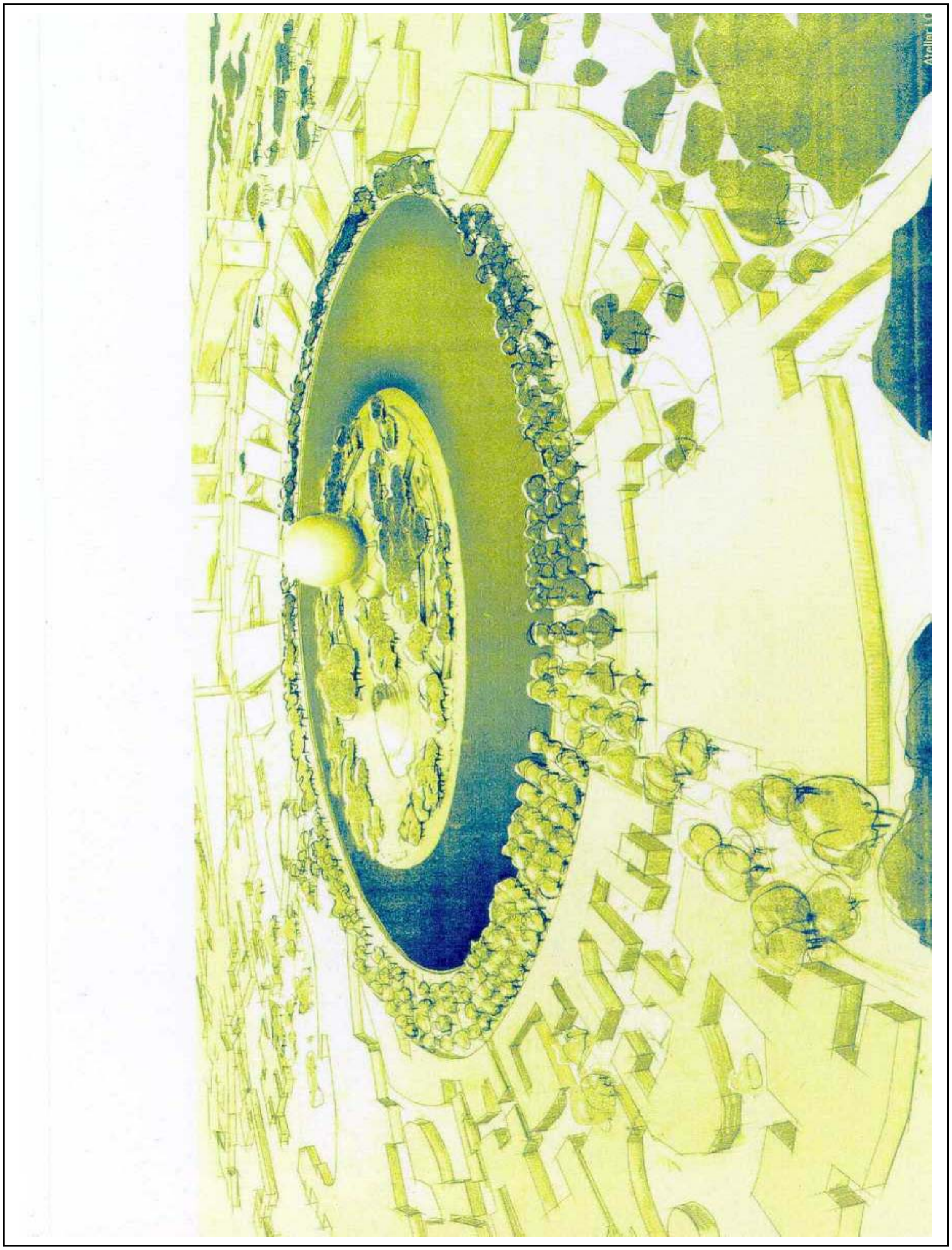


Figure 3.3: Matrimandir Lake (Visualization by H. Loidl)



4 PRE-FEASIBILITY STUDY FOR THE WATER SUPPLY OF THE CITY OF AUROVILLE

4.1 Water Resource Management

4.1.1 Introduction

The primary objective of storm-water harvesting as well as the treatment of wastewater for re-use is to develop a water resource management plan to ensure the availability of freshwater so that the needs of at Auroville are sustainably met. Over the coming decades, the management of water resources will become one of the most important issues across industrialized nations, as water availability and quality are likely to decrease. Given the already existing water problems encountered at Auroville, it is imperative that all sources of freshwater be considered, and if possible tapped to ensuring a safe and secure supply. According to the Water Management Concept it is proposed to recharge the first aquifer beneath the city with stormwater runoff that has been harvested from the city, and to supply the required drinking water to the city from the recharged aquifer. The groundwater should be extracted through a series of wells located in the GreenBelt. For the distribution of drinking water a common piped network is proposed which will be fed through booster pumps from one water works located in the GreenBelt.

Approximately 50 % of the total water demand will be met from rainwater harvesting from the roof tops of the buildings, and the other 50 % will have to be met from the groundwater resources. Since it is known that there is a high fluctuation in precipitation, and the supply from harvested rainwater may fall short in the future during dry years, the entire system has to be prepared so that the entire water demand can be met from the groundwater resource in times of need.

The recycling of water through the treatment and re-use of wastewater can make a positive contribution to the sustainability of available water resources. Central to the approach of water recycling is the concept of the utility of water, whereby water used is of a quality commensurate with its application. This then permits the exploitation of large water resources that are not necessarily of the highest purity. Moreover, domestic sewage carries substantial amount of valuable nutrients, among which nitrogen (N) and phosphorous (P) are dominant. Therefore a nutrient-rich treated wastewater can be conveniently used for irrigation purposes.

To conclude, it is proposed to meet the entire demand for irrigation out of the recycled wastewater. For peak demands that exceed the possible supply of wastewater suitable for re-use, groundwater has to be used and distributed through the irrigation pipe network (see drawing 42.02/1.3.7).

4.1.2 Water Demand

4.1.2.1 Population

According to the Master Plan, Auroville is separated into 6 zones with different water demands (see drawing 42.01/1.1.1).:

Residential Zone	40,000 inhabitants
International Zone	600 inhabitants
Industrial Zone	1,800 inhabitants
Cultural Zone	600 inhabitants
City Centre	5,000 inhabitants
Green Belt	2,000 inhabitants
Total	50,000 inhabitants

Additional demand results from external users in the Cultural and International Zone and from visitors. The water demand from commercial organizations, such as hotels and restaurants as well as from other non-commercial activities related to cultural, sporting and other events and functions also have to be considered. The specific water demand of the Industrial Zone has to be estimated from the proposed industries.

Additional water demand for commercial or industrial use can be estimated as follows:

Residential Zone	500 PE (Population Equivalent)
International Zone	1,500 PE (Population Equivalent)
Industrial Zone	10,000 PE (Population Equivalent)
Cultural Zone	3,500 PE (Population Equivalent)
City Centre	1,500 PE (Population Equivalent)
Total	17,000 PE (Population Equivalent)

The water demand of Auroville has to be estimated for a population of 50,000 and 17,000 PE (Population Equivalent) giving a total of 6,.000 P/PE (see Annex 1.1).

4.1.2.2 Drinking Water Demand

Drinking water demand of each inhabitant or population equivalent is estimated according to the general daily water consumption for different uses. This is generally known to be:*

Cooking and drinking	3-5 l/cap.d
Dish washing	10-20 l/cap.d
Personal hygiene	10-15 l/cap.d
Shower, bath	50-100 l/cap.d
Toilet flushing	40-50 l/cap.d
Laundry	15-30 l/cap.d
Garden, car	9-15 l/cap.d
Total	147-250 l/cap.d

Since it is assumed that Auroville will become an ecologically conscious city, where water will be used carefully, the water consumption is estimated to be on the lower side. All further estimates are based on the water consumption of

150 l/person per day

The daily water consumption can therefore be estimated to be

67,000 PE x 150 l/PE = 10,050 m^3/d.

The annual water consumption can thus be estimated to be

365 d x 10,050 m^3/d = 3.668,250 m^3/a.

Water for

Cooking and drinking	3-5 l/cap.d
Dish washing	10-20 l/cap.d
Personal hygiene	10-15 l/cap.d
Shower, bath	50-100 l/cap.d
Total	73-140 l/cap.d

has to be met from water of drinking quality.

Water demand for

House cleaning	10-15 l/cap.d
Laundry	15-30 l/cap.d
Total	25-45 l/cap.d

can be met from the use of harvested rainwater collected from roof tops.

* „Taschenbuch der Wasserwirtschaft“, Page 30

Water demand for

Toilet flushing	40-50 l/cap.d
Gardening, car washing	9-15 l/cap.d
Total	49-65 l/cap.d

can be met from the use of wastewater or harvested stormwater runoff.

Since the re-use of treated wastewater for flushing toilets would require additional pipelines in all households, and as the content of salts in the effluent would tend to increase as a result of re-circulating the same wastewater, it is recommended that only harvested rainwater should be used in those households.
Under these circumstances, it is therefore preferable to restrict the re-use of treated wastewater to irrigation purposes only.

The following estimates are based on the use of groundwater for:

Cooking and drinking	5 l/cap.d
Dish Washing	10 l/cap.d
Personal hygiene	10 l/cap.d
Shower, bath	50 l/cap.d
Total	75 l/cap.d

The daily and annual groundwater demand for drinking water supply can therefore be estimated to be:

67,000 PE x 0,075 m³/d = **5,025 m³/d**
5,025 m³/d x 365 d = **1,834.125 m³/a**

4.1.2.3 Demand for Harvested Rainwater

Harvested rainwater from the roof tops can be used to meet the following demand:

Toilet flushing	40 l/cap.d
House cleaning	10 l/cap.d
Laundry	15 l/cap.d
Total	65 l/cap.d

The daily and annual demand for harvested rainwater can be estimated to be (see Annex 1.4):

67,000 PE x 0,065 m³/d = **4,355 m³/d**
4,355 m³/d x 365 d = **1,589.575 m³/a**

4.1.2.4 Demand for Re-use of Treated Wastewater

Treated and recycled wastewater can be re-used to meet the demand for irrigation of personal gardens at a rate of

10 l/cap.day.

The daily and annual demand for treated and reused wastewater for irrigation at the premises can be estimated to be:

$$67.000 \text{ PE} \times 0{,}01 \text{ m}^3/\text{d} \quad = \quad 670 \text{ m}^3/\text{d}$$
$$670 \text{ m}^3/\text{d} \times 365 \text{ d} \quad = 244.550 \text{ m}^3/\text{a}$$

4.1.2.5 Irrigation

The green space in Auroville which requires irrigation can be identified as:

- private gardens
- green spaces in residential areas
- public green spaces, parks
- outer green spaces in the GreenBelt, agricultural areas

The green spaces in the city and in the greenbelt which could require irrigation have been estimated as below:

Table 4.1: Green Space in the City and in the Green Belt

Catchment Area for [--]	City Public Green Space [ha]	City Gardens [ha]	Outer Green Space [ha]
City North	50	69	0
City South West	25	43	0
City South East	6	15	0
City East	23	49	0
City Area	**104**	**176**	**0**
GreenBelt I	0	0	84
GreenBelt II	0	0	228
GreenBelt III	0	0	198
GreenBelt IV	0	0	235
GreenBelt V	0	0	76
GreenBelt VI	0	0	127
GreenBelt VII	0	0	258
GreenBelt Area	**0**	**0**	**1.206**
Total	**104**	**176**	**1.486**

In the city of Auroville 104 ha of public green spaces and 175 ha of garden area require irrigation. In the GreenBelt 1,206 ha of outer green space requires irrigation.

The irrigation demand has been estimated for a wet, a dry and an average year on the basis of the potential evapo-transpiration (PET) as well as the actual evapo-transpiration (aET) at 50 % of the potential evapo-transpiration according to table 4.1 and Annex 1.2.

Table 4.2: Irrigation Demand

month	precipitation average year * [mm]	precipitation wet year [mm]	precipitation dry year [mm]	potential evapo-trans-piration PET** [mm]	actual evapo-trans-piration aET*** (50%) PET [mm]	irrigation demand average year for PET [mm]	Irrigation demand wet year for PET [mm]	irrigation demand dry year for PET [mm]	irrigation demand average year for PET (50%) [mm]	irrigation demand wet year for PET (50%) [mm]	irrigation demand dry year for PET (50%) [mm]
January	36	88	25	114	57	78	26	89	21	-31	32
February	18	48	18	122	61	105	74	104	43	13	43
March	19	0	0	150	75	130	150	150	55	75	75
April	22	47	0	153	77	131	106	153	54	30	77
May	45	1098	38	195	98	150	-903	157	53	-1000	60
June	45	67	17	211	106	166	144	194	60	39	89
July	68	28	26	170	85	102	142	144	17	57	59
August	118	157	56	159	80	42	2	103	-38	-77	24
September	145	139	5	123	62	-22	-16	118	-83	-78	57
October	263	367	202	118	59	-145	-249	-84	-204	-308	-143
November	350	234	25	86	43	-264	-148	61	-307	-191	18
December	162	331	214	97	49	-65	-234	-117	-113	-282	-165
Total	**1293**	**2604**	**626**	**1700**	**850**	**407**	**-904**	**1074**	**-443**	**-1754**	**224**
					irrigation demand	903	645	1275	303	213	532
					infiltration	-496	-1549	-201	-746	-1967	-308

* ... Regional weather station, Pondicherry (1911-1971,1984-1991) ;Auroville - Certitude (1972-1983) ;Public Works Department, Pondicherry (1992-1995) ; Auroville-Aurodam (1996,1997) ; Auroville-Harvest (1998-2001)

** ... Potential Evapo-transpiration PET at Cuddalore (1981 to 1983)

***... Actual Evapo-transpiration aET is estimated at 50 % of Potential Evapo-transpiration PET

The irrigation demand for the entire green space of 1,485 ha has been estimated as follows in Table 4.3.

Table 4.3: Irrigation Demand at Actual and Potential Evapo-transpiration

Year [--]	irrigation demand at ET (50 % of PET) [m³/a]	irrigation demand at PET [m³/a]
average year	4,504,472	13,411,105
wet year	3,166,374	9,580,058
dry year	7,06,406	18,931,417

In an average year the irrigation requirement ranges between 4,5 M m³ and 13,4 M m³.
In the wet year the irrigation demand is reduced to 3,1 M m³ up to 9,6 M m³. The highest demand has to be met in a dry year. It ranges between 7,9 M m³ and 18,9 M m³.

4.1.3 Water Resources

4.1.3.1. Roof Top Rainwater Harvesting

The roof top area in the city of Auroville has been estimated to be 107 ha as indicated in table 4.4 (see Annex 1.5).

Table 4.4: Roof Top Area

catchment area [--]	city sector area [ha]	roof top area [ha]
city north	250	53,55
city south west	110	21,30
city south east	40	7,50
city east	120	24,30
city area	520	106,65

The runoff from the roof tops can be harvested in cisterns for direct re-use in the households. The runoff factor can be estimated to be μ = 0,9.

The precipitation has been estimated according to table 4.5.

Table 4.5: Precipitation in the City of Auroville

month [--]	precipitation average year [mm]	precipitation wet year [mm]	precipitation dry year [mm]
January	36	88	25
February	18	48	18
March	19	0	0
April	22	47	0
May	45	1,098	38
June	45	67	17
July	68	28	26
August	118	157	56
September	145	139	5
October	263	367	202
November	350	234	25
December	162	331	214
Total	**1,293**	**2,604**	**626**

The harvested water from the roof tops can be estimated according to table 4.6.

Table 4.6: Required Cistern Storage Volume

Year [--]	rainwater runoff from roof tops [m³/a]	required cistern storage volume [m³]
average year	1,227,648	469,977
wet year	1,858,941	845,829
dry year	852,l923	279,386

(see Annex 2.7.1,2,3)

4.1.3.2. Wastewater Re-use

The population of the city of Auroville is projected to be 50,000. An additional wastewater, equivalent to the use of 17,000 extra people, will be produced at commercial or industrial units. The daily wastewater generated per inhabitant or population equivalent is estimated to be 150 l/cap.·day. The annual wastewater production in the city can be estimated to be

$$67{,}000 \text{ PE} \times 0{,}15 \text{ m}^3/\text{PE}\cdot\text{d} \times 365 \text{ d/a} = 3{,}668.250 \text{ m}^3/\text{a}.$$

After purification the treated wastewater can be re-used to meet the water demand for irrigation. The losses of approximately 5 % for leakage and of 5 % for evaporation have been considered in the water balance.

4.1.4 Water Balance

Table 4.7: Water Balance in an Average Year

Annual water demand		Annual water resource	
[--]	[m³/a]	[--]	[m³/a]
drinking water supply	1,834,125	Groundwater	1,834,125
harvested rainwater reuse	1,589,575	harvested rainwater	1,195,850
Irrigation	4,504,472	wastewater reuse	3,368,250
Total	**7,928,172**		**6,398,225**

Water Balance	
	[m³/a]
drinking water supply	± 0
harvested rainwater re-use	-393,726
Irrigation	-1,136,222
Total	**-1,529,948**

The drinking water supply can be fully met by groundwater extraction. In a year with average rainfall, the harvested rainwater is not sufficient to meet the demand and 393,726 m³ have to be supplied through the drinking water supply from groundwater.
The irrigation demand, based on the actual evapo-transpiration (50 % of PET) in an average year, cannot be fully met by the re-use of wastewater and at balance of 1,136,222 m³ annually which would enable the irrigation of 75 % of the actual evapo-

transpiration aET (50 % of PET). The groundwater extraction has to be annually 3,364,073 m³.

In a wet year the precipitation exceeds the irrigation demand and 1,321,119 m³ can be used for groundwater recharge through infiltration annually.

Table 4.8: Water Balance in a Wet Year

Annual water demand		Annual water resource	
[--]	[m³/a]	[--]	[m³/a]
drinking water supply	1,834,125	Groundwater	1,834,125
harvested rainwater reuse	1,589,575	harvested rainwater	2,408,818
Irrigation	3,166,374	wastewater re-use	3,368,250
Total	**6,590,074**		**7,611,193**

Water Balance	
	[m³/a]
drinking water supply	± 0
harvested rainwater re-use	819,243
Irrigation	201,876
Total	**1,021,119**

In a wet year the drinking water demand can be met from groundwater. The demand for harvested rainwater for re-use can be fully met and 52 % of the demand 819,243 m³ can be annually infiltrated into the groundwater for groundwater recharge. In a wet year the precipitation exceeds the potential evapo-transpiration. Assuming a maximum water consumption by the plants at 50 % of the potential evapotranspiration, 29,251,141 m³ of the rainfall can be infiltrated for groundwater recharge (see Annex 1.3).

Since the irrigation demand cannot be fully met from precipitation additional water has to be supplied from treated wastewater. Only 3,166,374 m³ of treated wastewater are required for irrigation, the remaining amount of 201,876 m³ treated wastewater can be supplied to the neighbouring communities for irrigation or infiltration into an aquifer which is not used for drinking water supply. Before infiltration into the first aquifer used by Auroville the full de-salinisation through reverse osmosis would be required to protect the aquifer. Re-mineralisation can be expected through the filtration within the aquifer. The groundwater extraction in a wet year can therefore be limited to the drinking water supply of annually 1,834,125 m³.

Table 4.9: Water Balance in a Dry Year

Annual water demand		Annual water resource	
[--]	[m³/a]	[--]	[m³/a]
drinking water supply	1,834,125	Groundwater	1,834,125
harvested rainwater re-use	1,589,575	Harvested rainwater	579,584
Irrigation	7,906,406	Wastewater re-use	3,368,250
Total	**11,330,106**		**5,781,959**

Water Balance	
	[m³/a]
drinking water supply	± 0
harvested rainwater re-use	-1,009,992
Irrigation	-4538,156
Total	**-5,548,148**

In a dry year the drinking water supply can be met from groundwater. The demand for harvested rainwater for re-use can only meet up to 36 % of the total demand. The deficit of 1,009,992 m³ annually have to be supplied from groundwater through the drinking water supply system. The water demand for irrigation can only meet up to 43 % and 4,538,156 m³ have to be supplied annually from groundwater. The total annual groundwater extraction would have to be 7,382,273 m³ where 2,844,117 m³ have to be drinking water quality.

The harvested roof top rainwater and the treated rainwater as water resource for domestic water supply and irrigation cannot meet the water demand at all times, so the balance has to come from the groundwater. The groundwater extraction can therefore be estimated according to table 4.10 (see Annex 1.6).

Table 4.10: Demand for groundwater extraction for the domestic and irrigation water supply*

water supply	average year	dry year	wet year
[--]	[m³/a]	[m³/a]	[m³/a]
drinking water supply**	1,834,125	1,834,125	1,834,125
harvested rainwater re-use	393,726	1,009,992	--
Irrigation	1,136,222	4,538,156	--
Total	**3,364,073**	**7,382,273**	**1,834,125**

* for actual evapo-transpiration (aET = 50 % of PET)
**for the supply of 75 l/ capita

4.2 Groundwater Resource

4.2.1 Present Water Supply

The water supply at present is met from two aquifers. The first aquifer in the Cuddalore Sandstone at a depth of 20-50 m is in the Auroville area. It is above sea level, has a very low permeability, and has wells with very low yields (see Fig. 4.1, 4.2, 4.3)

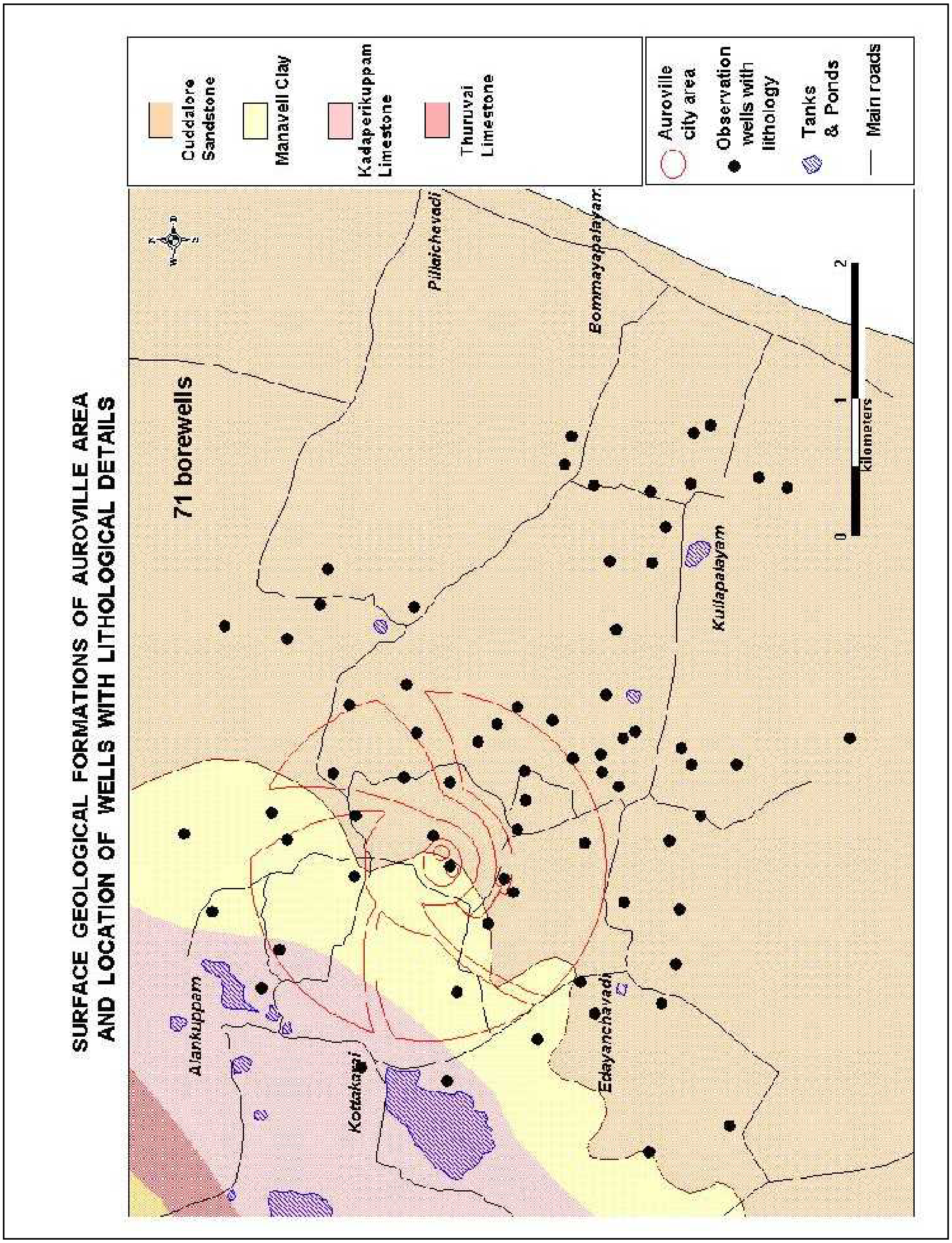
The second aquifer is in the Kadaperikuppam Limestone in a layer 10 to 50 m thick 0 to 50 m below sea level. The limestone at present has wells with high yields because of its high permeability and most of the Auroville wells are drawing water from this source. (see Fig. 4.4, 4.5, 4.6). At present 190 wells have been sunk in Auroville mainly tapping the second aquifer.

Over- extraction of groundwater in the region from this aquifer has lead to salt water intrusion that has been observed in the Cuddalore Sandstone at a lower level towards the Kalivelli Tank. (see Fig. 2.4, 2.5, 2.6, 2.7, 2.8)
At present within the proposed city area 150 wells are provided, out of the total of 190 under the control of Auroville.

4.2.2 Proposed Water Supply from Groundwater Source

The first aquifer located beneath Auroville slopes in a south-easer direction towards the sea. It remains above sea level and is therefore free from salinisation by seawater intrusion.

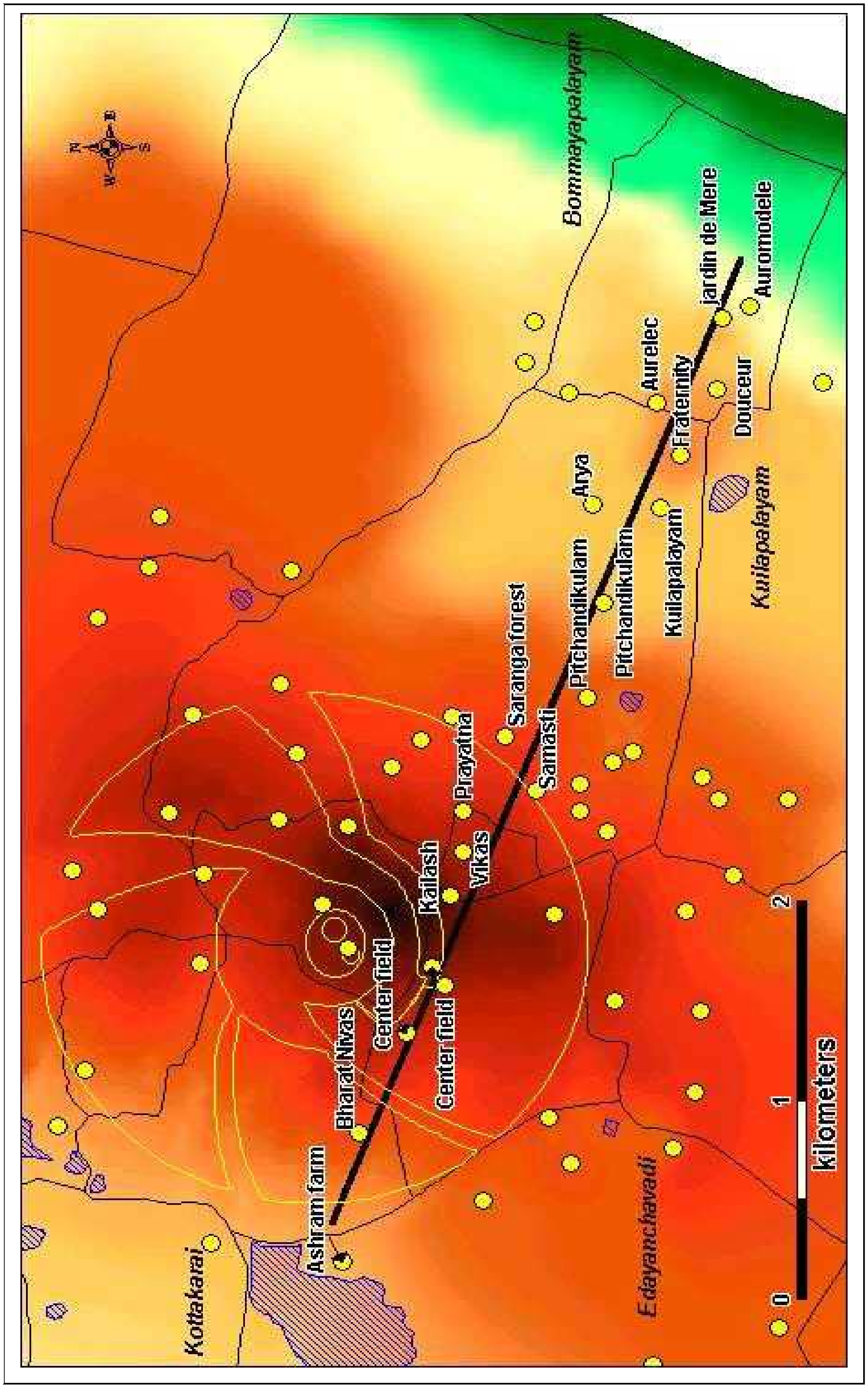
Figure 4.1: Surface Geological Formations of Auroville Area and Location of Wells with Lithological Details



(Source: Harvest, Interim Report on Auroville's Water Conditions)

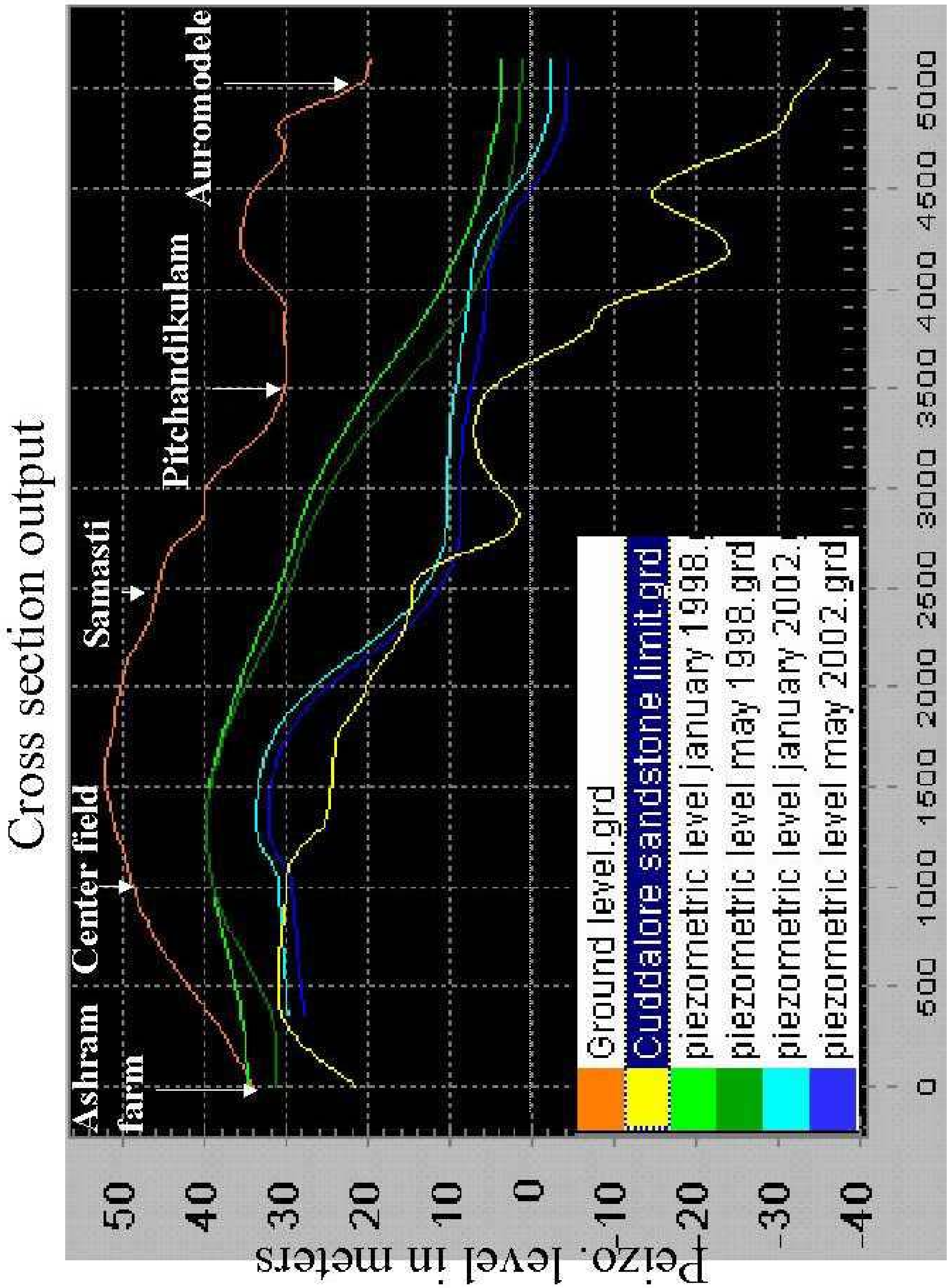
Figure 4.2: Line selected for cross section

Line selected for cross section



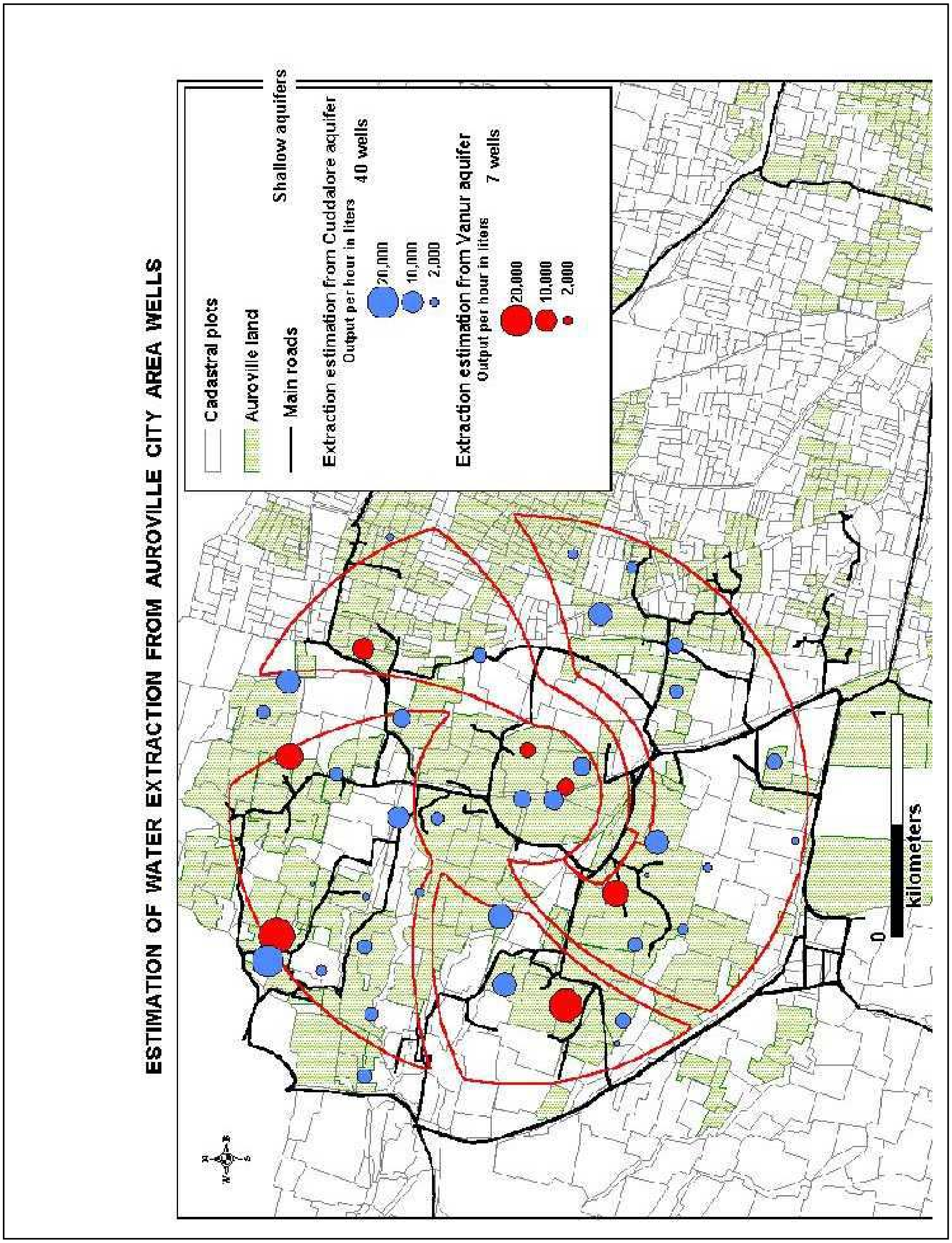
(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 4.3: Cross Section Output



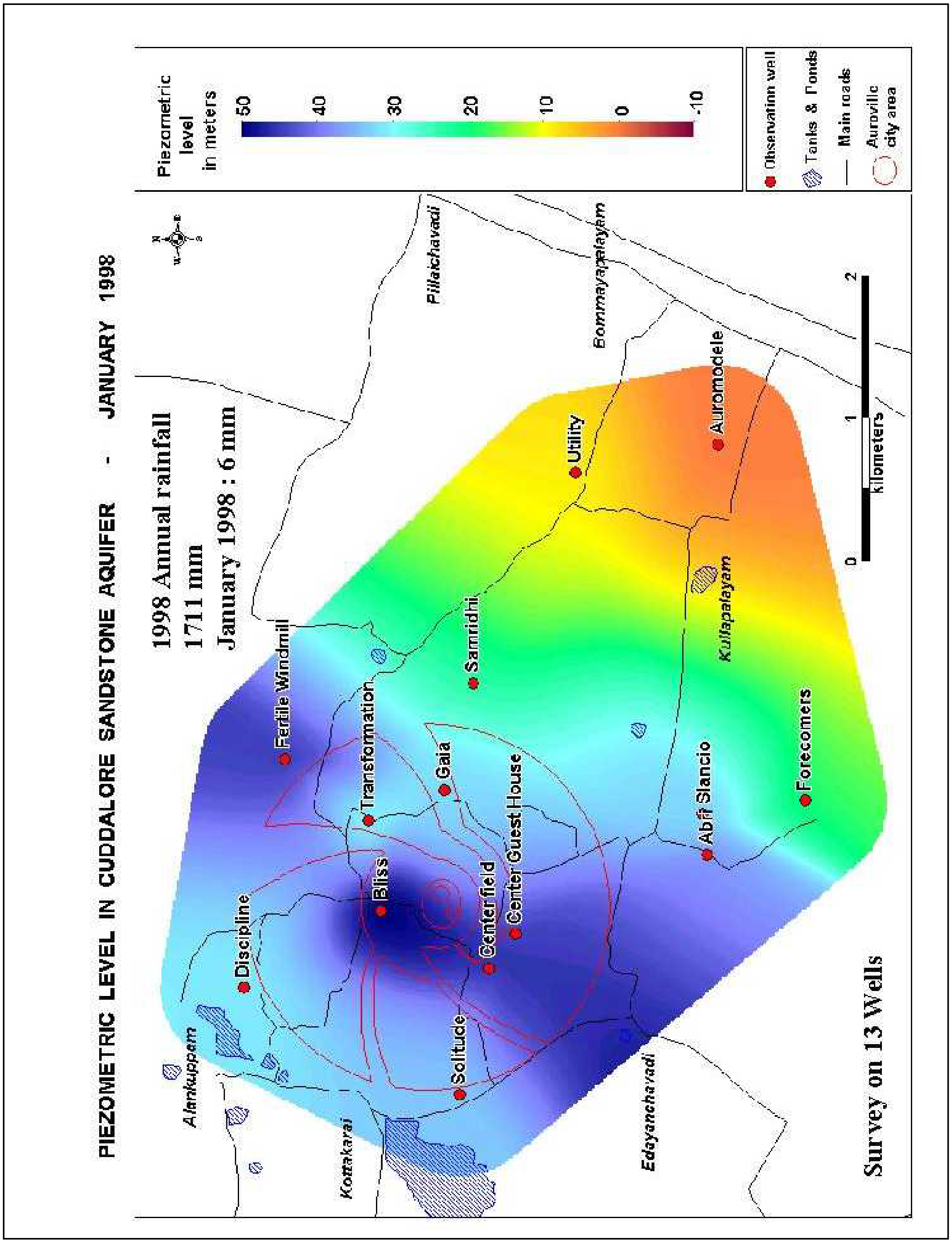
(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 4.4: Estimation of Water Extraction from Auroville City Area Wells



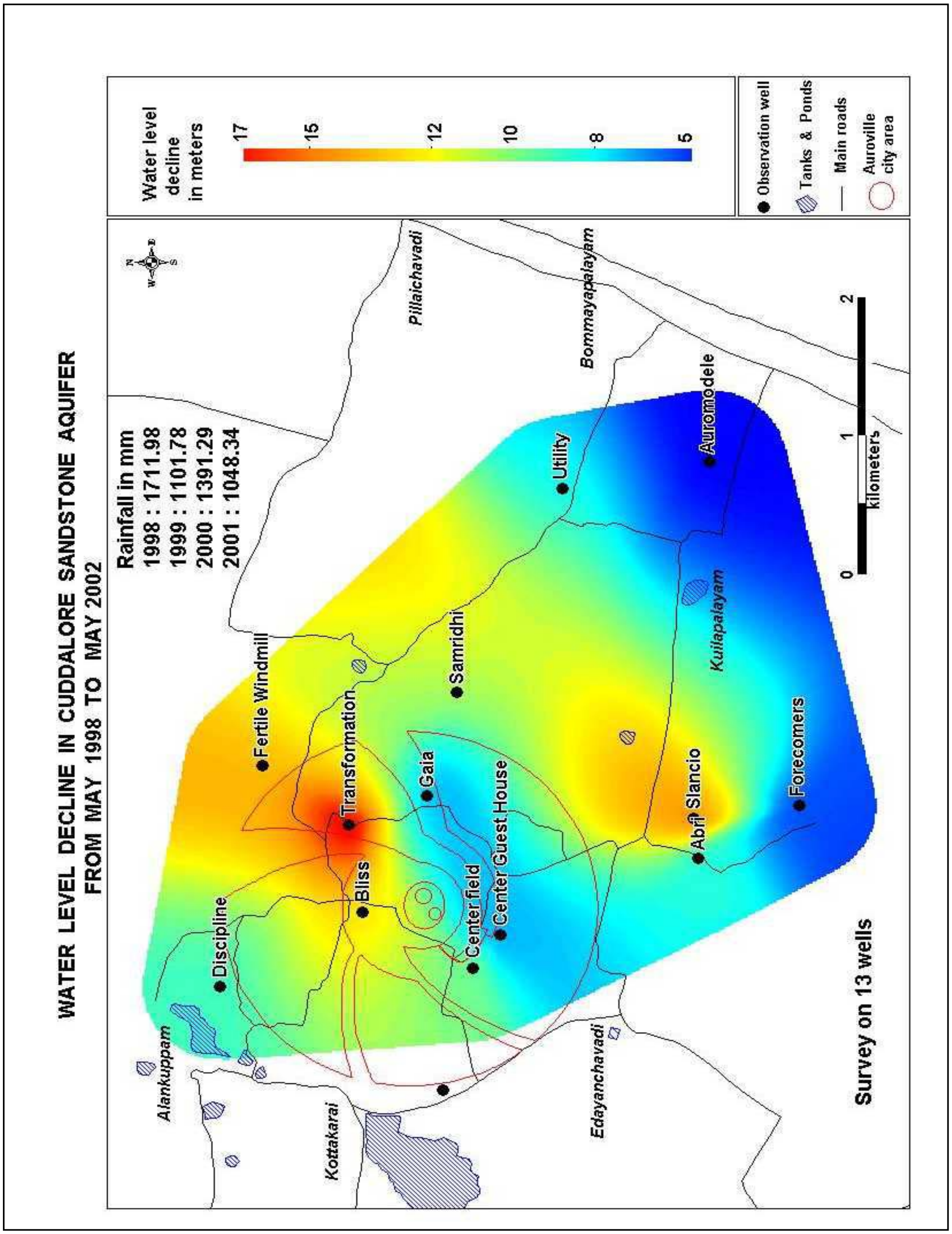
(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 4.5: Piezometric Levels in Cuddalore Sandstone Aquifer – January 1998



(Source: Harvest, Interim Report on Auroville's Water Conditions)

Figure 4.6: Water Level Decline in Cuddalore Sandstone Aquifer From May 1998 to May 2002



(Source: Harvest, Interim Report on Auroville's Water Conditions)

The capacity of the first aquifer has to be increased by groundwater recharging so that it can meet the requirement for groundwater extraction. For this purpose it is proposed to harvest the stormwater runoff from the entire city area and infiltrate it in the Cuddalore Sandstone into the first aquifer at the city centre after it has undergone treatment . (see Figure 4.1, 4.2, 4.3)

The entire groundwater in this aquifer moves towards the sea on top of the Manaveli clay. The recharged groundwater can be extracted in borewells located in the GreenBelt just before the limits of the city. After providing a minimum treatment by rapid sand filtration and chlorination, the water can be supplied to the city through a drinking water network with booster pumps.

4.2.3 The Cuddalore Sandstone Aquifer

It is proposed to use the first aquifer beneath the city to store the required drinking and irrigation water that is to be supplied to the city.
The underground storage of water in the aquifer should also balance the annual variation of precipitation as well as the long-term variation over the years, so that the excess of harvested rainwater from a wet year can be used to make up for the deficit of rainwater during a dry year. The sustainable use of this aquifer is of most importance for the future water supply of the city. The depth of the Cuddalore Sandstone within the Auroville area is approximately 30 m.

As the “Water Resource Study for Auroville” has not been completed by the scheduled date of January 2002, and essential information related to the groundwater flow and the Cuddalore Sandstone Aquifer, such as transmissivity from pump tests, have not been made available so far, the Pre-Feasibility Study has to be based on the available information on the lithology and hydrology from the hydro-geological study of 1984.

Table 4.11: Results of Pumping Tests – Transmissivity from 1984

No.	Tubewell No.	Location	Date of Test	Depth [m]	Aquifer	Transmissivity [m³/d·m]
1	K 1	Forecomers	5-3-84	76.2	Sandstone + Limestone	5.6
2	I 16	Field (Mr. Michael)	11-3-84	34.0	Sandstone	22.0
3	B 5	New Creation Fraternity	12-3-84	66.0	Sandstone + Limestone	177.0
4	I 21	Sincerity	13-3-84	48.0	Sandstone + Limestone	26.0

The transmissivity varies significantly, probably because of the change in the lithology since the wells may have reached the limestone. The transmissivity of the Cuddalore aquifer was given at 9,26·E-06 m/s (0,8 m/d) up to 3,4·E-0,5 m/s (3,0 m/d) (Source: Hydro -geological Conditions in Auroville, May 1984). With a velocity of 3,0 m/d, a given drop of water would require 417 days to travel the distance from the proposed recharge point to the city centre limit, and the same time to reach the limit of the GreenBelt. Thus the retention time of the first aquifer would be 2,3 years. Considering the lower velocity of 0,8 m/d, the retention time of a given drop of water would be 8,6 years.

For this water management concept, the minimum required retention time is 1 year. With such a retention time a given drop of water would have a velocity of 6,9 m/d [7,99·E-5 m/s]. The available data from the literature and from the test carried out by Mr. Schlenther in February 2002 do not indicate that the high groundwater flow velocities of 6,9 m/d [7,99·E-5 m/s] can be obtained. (Study on the infiltration capacity in Auroville, 2002). As transmissivity data is vital for this concept, it is recommended that further studies be carried out in this respect ((see drawing 42.01/1.3.2).

During the various visits to Auroville and India before and during this study it was found that there is widespread belief that the underground environment (below the topsoil) contributes significantly to the purification of polluted water infiltrated into an aquifer. In Germany, among hydrogeologists, there is a thumb rule, that it is safe to extract groundwater after a filtration time of 50 days. This rule is generally based on alluvial aquifers and cannot be compared with aquifers located in sandstone or limestone.

The mortality of intestinal micro - flora is high if it is discharged into an aquifer. Especially the indicator bacteria for fecal pollution like coliforms and Escherischia Coli will be removed efficiently. But very little is known on the survival of pathogenic bacteria in groundwater, and less is known about the survival of pathogenic viruses. Through coarse materials (like sandstone or limestone), pathogens can travel long distances. It is known that bacteria will live longer at lower temperatures, neutral pH-value, in the absence of sunlight and bacteriophages, and if organic substances are available in the infiltrated water. E-Coli have been detected in groundwater even after 1,000 days. Entero bacteria have been found to survive more than 5 years in soil.

The survival of the viruses in the groundwater depends very much on the temperature. Low temperatures increase the survival time. Entero-viruses have been found in fresh water even after 188 days. Viruses that have been absorbed to soil particles are known to be as pathogenic as those in the water. Polio viruses and f2 viruses have been found to survive in sandy soil for periods up to 175 days at low temperatures. The presence of organic substances with the infiltrating water increases the life time of the pathogens in the ground. Organic substances may also lead to the growth of bio-films in the soil which may reduce permeability. From the above it becomes obvious that for the protection of a sustainable aquifer it is best to treat it like a fresh water storage basin rather than using it as a treatment plant, as once it has got polluted it is almost impossible to clean.

The water that is infiltrated into the aquifer should have almost drinking water quality. The water used to recharge the aquifer should contain almost no filterable organic substances and very little dissolved organic substances, and should have a very low concentration of pathogens.

4.3 Groundwater Extraction

From the groundwater recharge area located in the Peace Area of the Auroville at the Centre of the town, the aquifer will be built up and used for underground storage of harvested rainwater. It is proposed to extract the groundwater from the Cuddalore Sandstone aquifer on the borders of the GreenBelt.

4.3.1 Dimensioning of the Groundwater Extraction Wells

The inflow of a well is calculated on the basis of the continuity equation:

$$Q = v \cdot A$$

and the Darcy-equation

$$Q_f = k_f \cdot I \cdot A \text{ .}$$

The inflow of the well is calculated with the following equation:

$$Q_z = (H^2 - h_{br}^2) \cdot \frac{\Pi \cdot k_f}{\ln R / r} \qquad [m^3/s]$$

R is the empirically determined range of the well according to Sichardt:

$$R = 3000 \cdot s \cdot \sqrt{k_f}$$ and

H	=	depth of the aquifer (in m)
h_{br}	=	depth of the lowered groundwater table within the well (in m)
r	=	radius of the well (in m)
s	=	height of the depression of the groundwater table (in m)
k_f	=	permeability (in m/s) of aquifer.

The maximum capacity of the well Q_f is calculated on the basis of the continuity equation

$$Q = v \cdot A$$

On the rim of the well, v will be v_{krit} according to Sichardt:

$$v = \frac{\sqrt{k_f}}{15}$$

With this Qf is calculated as:

$$Q_f = \frac{2}{15}\Pi \cdot r \cdot h_f \cdot \sqrt{k_f} \qquad [m^3/s]$$

Because of the inconsistency of the flow towards the well Qf will be reduced to Qf/2

$$Q_f = \frac{\Pi}{15} r \cdot h_f \cdot \sqrt{k_f} \qquad [m^3/s]$$

From the Hydro- geological Study and from the study on the infiltration capacity the permeability k_f and the depth H of the aquifer in the Cuddalore Sandstone was estimated to be:

k_f = $3{,}47 \times 10^{-5}$ m/s
H = 10 m

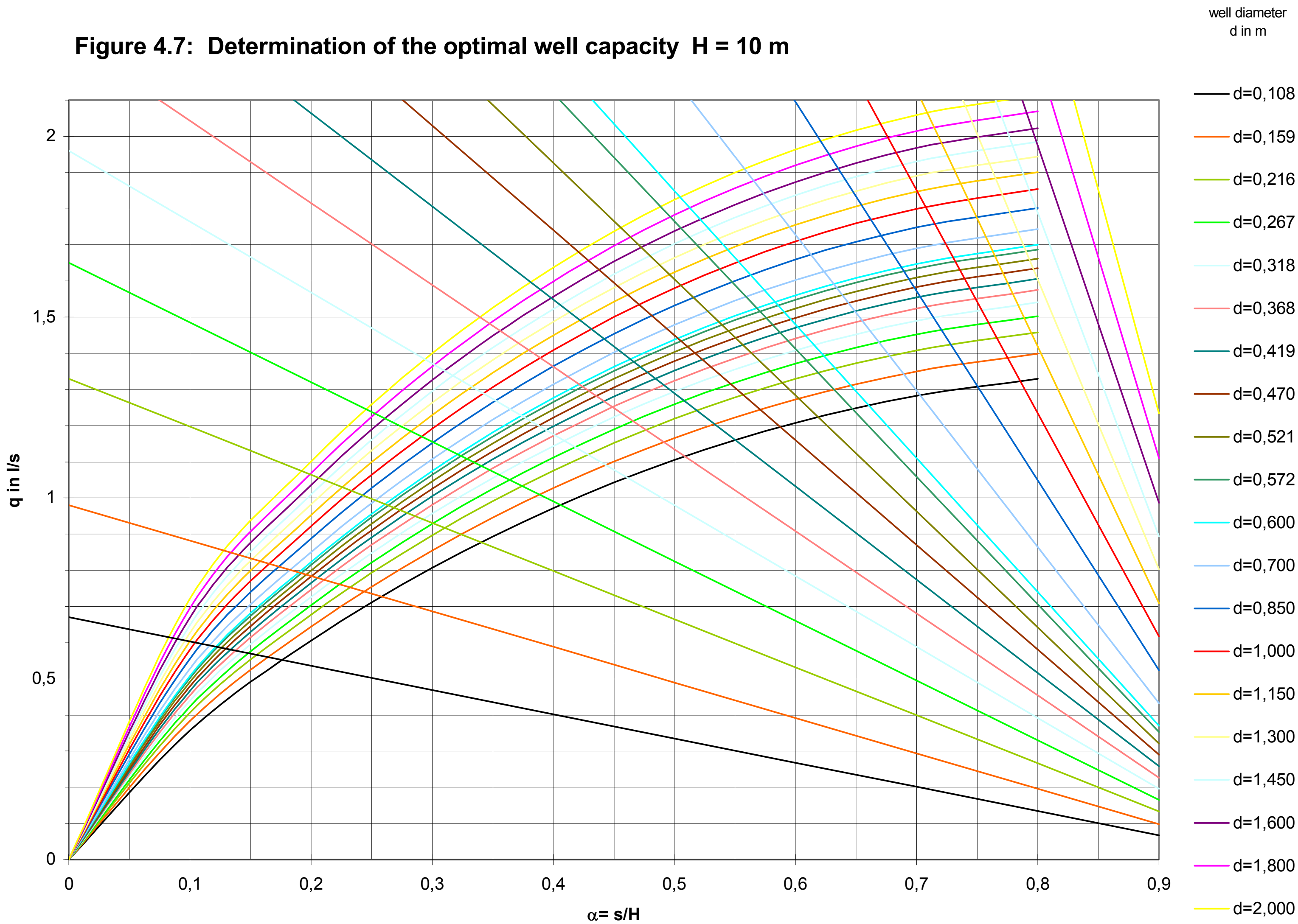
The capacity of the wells q_f is estimated according to the well diameter D at s = 0 as follows:

Table 4.12: Capacity q_f of the wells:

D	$q_{f\ (s=0)}$
[m]	[l/s]
0,108	0,67
0,159	0,98
0,216	1,33
0,267	1,65
0,318	1,96
0,368	2,27
0,419	2,58
0,470	2,90
0,521	3,21
0,572	3,53

D	$q_{f\ (s=0)}$
[m]	[l/s]
0,600	3,70
0,700	4,32
0,850	5,24
1,000	6,17
1,150	7,09
1,300	8,02
1,450	8,94
1,600	9,87
1,800	11,10
2,000	12,34

Figure 4.7: Determination of the optimal well capacity H = 10 m



The inflow to the well q_z is estimated with the following equation:

$$q_z = \Pi \cdot k_f \cdot \frac{(H^2 - h_{br}{}^2)}{\ln R / r} \quad \text{at}$$

$$q_z = \Pi \cdot k_f \cdot H^2 \frac{\alpha(2-\alpha)}{\ln R / r} \quad \text{and} \quad \alpha = \frac{s}{H}$$

with $R = 3000 \cdot s \cdot \sqrt{k_f} = 3000 \cdot \sqrt{k_f} \cdot H \cdot \alpha$

Table 4.13: Estimate of the Inflow to the Well Q_z

s [m]	1,5	3	4,5	6	7,5	9	10,5	12
α	0,1	0,2	0,3	0,4	0,5	0,6	0,7	0,8
R [m]	26,51	53,02	79,52	106,03	132,54	159,05	185,56	212,06
d [m]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]	Q_z [l/s]
0,108	0,75	1,28	1,71	2,07	2,36	2,58	2,74	2,85
0,159	0,80	1,36	1,81	2,18	2,48	2,71	2,88	2,98
0,216	0,85	1,43	1,89	2,28	2,59	2,82	3,00	3,11
0,267	0,88	1,48	1,96	2,35	2,67	2,91	3,08	3,19
0,318	0,91	1,52	2,01	2,41	2,74	2,98	3,16	3,27
0,368	0,94	1,56	2,06	2,47	2,80	3,05	3,23	3,34
0,419	0,96	1,60	2,11	2,52	2,85	3,11	3,29	3,40
0,470	0,99	1,63	2,15	2,57	2,90	3,16	3,35	3,46
0,521	1,01	1,66	2,19	2,61	2,95	3,21	3,40	3,51
0,572	1,03	1,69	2,22	2,65	3,00	3,26	3,45	3,56
0,600	1,04	1,71	2,24	2,68	3,02	3,28	3,47	3,59
0,700	1,08	1,76	2,31	2,75	3,10	3,37	3,56	3,68
0,850	1,13	1,83	2,39	2,84	3,20	3,48	3,67	3,79
1,000	1,17	1,89	2,47	2,93	3,30	3,58	3,77	3,89
1,150	1,22	1,95	2,54	3,01	3,38	3,66	3,86	3,98
1,300	1,26	2,01	2,60	3,08	3,46	3,75	3,95	4,07
1,450	1,29	2,06	2,66	3,15	3,53	3,82	4,03	4,15
1,600	1,33	2,11	2,72	3,21	3,60	3,89	4,10	4,22
1,800	1,38	2,17	2,79	3,29	3,68	3,98	4,19	4,31
2,000	1,42	2,22	2,86	3,37	3,76	4,06	4,27	4,40

The inflow to the well q_z has been calculated for different well diameters using different values of α $\left(\alpha = \frac{s}{H}\right)$ that provide a maximum and a minimum value. Compared with the capacity of the wells q_f and displayed in a diagram, the optimum for each well diameter can be determined by the intersection of $q_z = q_F$.

The optimal capacity of the wells from diameter of 0,108 m to 1,0 m are shown in the following table:

Table 4.14: Optimized well capacity q_{opt}

Well diameter D [m]	Optimized well capacity (as shown in diagram) $q_{opt.}$ [l/s]
0,108	0,55
0,159	0,74
0,216	0,92
0,267	1,05
0,318	1,16
0,368	1,25
0,419	1,33
0,470	1,40
0,521	1,46
0,572	1,52
0,600	1,54
0,700	1,63
0,850	1,73
1,000	1,81

According to the optimized capacity of the wells, the number of wells and their diameter and depth can be chosen to meet the requirement for the full supply of drinking water for the city of Auroville.

Table 4.15: Estimate of the required number of wells for the drinking water supply of Auroville

Inhabitants		67.000 P / PE
Water Consumption per Person	q_{PE}	**100 l/d**
Thickness of the Aquifer	H:	10 m
Permeability of the Aquifer	k_f	3,47E-05 m/s

Diameter of Well D	Optimized Well Capacity $q_{opt.}$	Drinking Water Demand Q_{ges}	Required Number of Wells	Selected Number of Wells	From Diagram derived α	Range of the Groundwater depreciation R	Total length of the Well Field l
[m]	[l/s]	[l/s]				[m]	[m]
0,108	0,55	77,55	140,99	141,00	0,177	31,28	8820,81
0,159	0,74	77,55	104,79	105,00	0,245	43,30	9092,25
0,216	0,92	77,55	84,38	85,00	0,310	54,78	9313,15
0,267	1,05	77,55	73,85	74,00	0,365	64,50	9546,42
0,318	1,16	77,55	66,85	67,00	0,410	72,46	9709,00
0,368	1,25	77,55	62,04	63,00	0,450	79,52	10020,03
0,419	1,33	77,55	58,31	59,00	0,483	85,36	10071,99
0,470	1,40	77,55	55,39	56,00	0,515	91,01	10193,22
0,521	1,46	77,55	53,00	54,00	0,545	96,31	10401,75
0,572	1,52	77,55	51,02	52,00	0,570	100,73	10475,97
0,600	1,54	77,55	50,35	51,00	0,583	103,03	10508,84
0,700	1,63	77,55	47,57	48,00	0,623	110,10	10569,28
0,850	1,73	77,55	44,82	45,00	0,670	118,40	10656,22
1,000	1,81	77,55	42,96	43,00	0,706	124,76	10729,74

Table 4.16: Estimate of the required number of wells for the drinking water supply of Auroville

Inhabitants		67,000 P / PE
Water Consumption per Person	q_{PE}	**150 l/d**
Thickness of the Aquifer	H:	10 m
Permeability of the Aquifer	k_f	3,47E-05 m/s

Diameter of Well D [m]	Optimized Well Capacity $q_{opt.}$ [l/s]	Drinking Water Demand Q_{ges} [l/s]	Required Number of Wells	Selected Number of Wells	From Diagram derived α	Range of the Groundwater depreciation R [m]	Total length of the Well Field l [m]
0,108	0,55	116,32	211,49	212,00	0,177	31,28	13262,49
0,159	0,74	116,32	157,19	158,00	0,245	43,30	13681,67
0,216	0,92	116,32	126,57	127,00	0,310	54,78	13914,94
0,267	1,05	116,32	110,78	111,00	0,365	64,50	14319,63
0,318	1,16	116,32	100,28	101,00	0,410	72,46	14635,96
0,368	1,25	116,32	93,06	94,00	0,450	79,52	14950,52
0,419	1,33	116,32	87,46	88,00	0,483	85,36	15022,62
0,470	1,40	116,32	83,09	84,00	0,515	91,01	15289,82
0,521	1,46	116,32	79,51	80,00	0,545	96,31	15409,99
0,572	1,52	116,32	76,53	77,00	0,570	100,73	15512,49
0,600	1,54	116,32	75,53	76,00	0,583	103,03	15660,23
0,700	1,63	116,32	71,36	72,00	0,623	110,10	15853,92
0,850	1,73	116,32	67,24	68,00	0,670	118,40	16102,74
1,000	1,81	116,32	64,44	65,00	0,706	124,76	16219,37

In an average year, the daily demand of 100 l/capita of drinking water can be met by wells with a diameter of 0,368 m. The daily balance of 50 l/capita can be provided by rainwater harvesting. The extreme variation of rainfall that has been observed in the past and that is predicted in the future may not allow the provision of sufficient harvested rainwater during every year. It is therefore proposed to dimension the entire drinking water supply system for emergency situations such as when sufficient harvested rainfall is not available, and when the entire water demand has to be met from the groundwater source. The dimension of the well field, the water works and the distribution network have to be calculated on the basis of the daily consumption of 150 l/capita.

The costs of the drilling and of the technical equipment in relation to the optimized utilization of the groundwater resource will determine the choice of the diameter of the wells. For the dimensioning of the required well field for the extraction of groundwater, the optimal diameter of the well has been selected from the table “Estimate of the required number of wells for the drinking water supply of Auroville”. With the use of wells with the chosen well diameter of D = 0,368 m and with a capacity of q_{opt} = 1,25 l/s the number of wells required to meet the average daily demand is 94. In order to meet the daily peak demand, an underground water storage sump is required. For the treatment of the extracted groundwater a simple rapid sand filtration has to be provided. The backwash would require 1 % to 2 % of the daily water consumption. Thus, the number of wells would have to be increased by two to a total of 96. To provide security in the supply of 150 l/capita of potable water in case of maintenance and repairs at some wells, the total number of required wells would be 100.

The annual yield from the well field of 100 wells can be estimated at

Q_{max} = 100 wells x 0,00125 m³/s x 3,600 s x 24 h x 365 d = 3,942,000 m³/a.

The well field has been designed for an annual capacity of

Q_a = 3,668,250 m³/a

According to the water balance in Table 4.10 the requirement for groundwater extraction in an average year is

Q_m = 3,364,073 m³

and in a wet year

Q_w = 1,834,125 m³.

This demand can be met by the proposed number of wells.

In a dry year the demand for domestic water is estimated to be

Q_{dd} = 2,844,117 m³

and the additional water demand for irrigation is estimated to be

Q_{di} = 4,538,156 m³

The total demand for groundwater extraction in a dry year would be

Q_d = 7,382,273 m³

which will be 201 % of the designed yield of the proposed 100 wells.

To cover the remaining demand for irrigation of

Q_{di} = 3,714,023 m³

An additional 100 wells of the proposed capacity have to be provided.

To meet the full water demand from groundwater extraction even in a dry year the provision of up to 200 wells is required.

At a radius of 2,4 km from the centre point in the western part of the GreenBelt, for the drinking water supply a well field of 60 wells can be located with each well being 160 m apart. The remaining 40 wells can be located in a well field located at a radius of 2,0 km from the centre point each 200 m apart. Each of the wells has to be provided with a casing and a filter pipe with gravel packing. The submersible pumps will be placed below the filter pipe. The well head will be constructed from pre -cast concrete elements (D = 2,5 m, H = 2,0 m), including a clay sealing towards the well. The technical equipment for electricity supply, valves, return valves, water meter and manometer as well as an alarm device will be fitted into the wellhead. All wells will be connected to a pipeline which will be linked to the water works. The 100 wells for the irrigation can be located at the centre of the demand or in areas of high yield within the GreenBelt (see Annex 1.7 and drawings 42.01/1.1.3).

4.4 Water Works

The water works is best located in the middle of the two well fields. In the present conditions the groundwater requires no treatment. In future this is bound to change since groundwater recharge and all human activities will reflect on the quality of the groundwater. It is therefore of utmost importance that the groundwater source, the first aquifer, is sustainable and protected. For the time being the proposed treatment given to the groundwater is simple rapid sand filtration and chlorination or ozonisation to disinfect the distribution systems.

The type of treatment technology to be used would finally have to be determined according to the quality of groundwater available when the full city is established, and when the full groundwater recharging as proposed above is undertaken.

4.4.1 Dimensioning of the Rapid Sand Filtration

The filter should have a depth of 2,000 mm, with filter material of quartz sand Ø 0,5 mm to 2,0 mm in a single layer. The filter velocity should be v = 11 m/h. For the backwash 1 % to 2 % of the daily filtered volume will be required.
The required filter surface has to be estimated for a maximum flow of Q = 113,3 l/s.

Q_t	= 116,3 l/s	
Q_{Return}	= 116,3 l/s x 0,02	= 2,33 l/s
Q_{SF}	= 118,633 l/s	= 427,08 m³/h
V	= 11 m/h	
A_{SF}	= Q / v	
A_{SF}	= 427,08 m³/h / 11 m/h	
A_{SF}	= 38,83 m²	

It is proposed to select closed, standing Rapid Sand Filters of 3 m diameter. The surface area of each filter with D = 3 m is:

A_S	$= \pi/4 \times d^2$
A_S	= 7,07 m²

Thus the required number of Rapid Sand Filters can be estimated to be

A_{SF} = 38,8 m² : 7,07 m² = 5,5 Filters.

After 20 to 100 hours of operation the filters require 15 to 20 minutes of backwashing at a speed of 60 – 90 m/h. Considering the backwash time and time for maintenance and repair the filtration of the groundwater requires 7 Nos. of closed Rapid Sand Filters each with a diameter of

D = 3,0 m

and height H = 4,5 m.

After the filtration process the water should be disinfected. For the disinfection of drinking water chlorination is still the most economic and simple technical solution (if the concentration of dissolved organics are low). Disinfection is considered safe if 0,1 mg of free chlorine is detected at the consumers' tap.
Ozonization can be considered as well for disinfection of the water. The final decision can only be made when the quality of the filtrate and the quality of the pipe network are known.

4.4.2 Dimensioning of the Underground Water Storage Tank

The peak loads during the day and different seasons should be equalized in a storage tank to minimize the size of the groundwater extraction facilities.
The average daily water supply can be estimated to be:

Q = 67,000 PE x 150 l/d / 24 h / 3600 s
Q = 116,3 l/s

The size of the water storage tank should be determined considering a state of emergency and should be sufficient for the water consumption of at least one day.

Q = 67,000 PE x 150 l/d / 1000 l/m³
Q = 10,050 m³

The dimensions of two circular underground water storage tanks are:

D = 36,0 m
H = 5,0 m.

4.4.3 Dimensioning of the Booster Pumps

The treated groundwater will be supplied to the city with the use of Booster Pumps. These pumps have to be designed for the peak load. The peak load factor is estimated to be 2,7.

Thus, the peak flow can be estimated to be:

Q_{Sp} = 67,000 PE x 150 l/d / 24h / 3600s x 2,7

Q_{Sp} = 314 l/s.

The required pressure is estimated on the basis of the geographical conditions and the frictional losses in the pipe network during the peak supply time.

The booster pumps have to deliver a pressure of

P = 11 bar.

To deliver the required designed supply during peaks, 6 pumps are required each with a capacity of 55 l/s. Every pump has to operate up to the maximum pressure but has to deliver only part of the total supply.

The operation of the pumps has to be automatically controlled by the water pressure in the pipe system. Water consumption will result in reduction of pressure in the pipe system, and when the pressure drops beneath the determined level the first booster pump will start operating. The first pump has to be operated by a Tyristo Control that automatically adjusts the pumping capacity in relation to the pressure in the pipe system. If the capacity of the pumps does not meet the demand, additional pumps have to be automatically started so that the pipe system operates under full pressure and full capacity. With this system booster pumps can optimally supply water according to the demand of water and very small balancing tanks are required and the operational costs are minimized as well (see drawing 42.01/1.2.4, 1.3.1).

4.5 Distribution network

4.5.1 Drinking water supply

The demand for drinking water has been estimated to be

q_d = 75 l/cap.d (see chapter 4.1.2.2).

The daily drinking water demand is estimated to be

Q_d = 5,025 m³/d

The annual drinking water demand has been estimated to be

Q_a = 1,834,125 m³/a.

This water demand will be met entirely from the groundwater source. From tubewells the groundwater will be pumped to the Water Works for treatment and from there booster pumps will feed the drinking water into the distribution network (see drawing 42.01/1.1.2).

4.5.2 Dimensioning of the Distribution Network

The Distribution Network has one intake at the Water Works.
The System is dimensioned for an intake of

Q_{max} = 314 l/s.

For the dimensioning of the pipe network the computer programme from the company Barthauer based on the Hardy Cross Calculation has been used.

The friction losses have been calculated using the equation of Darcy-Weissbach:

$$hr = \lambda \cdot \left(\frac{l}{d} \cdot \frac{v^2}{2 \cdot g}\right)$$

with $$v = \frac{Q}{A}$$

The pressure losses can be derived from:

$$hr = \lambda \cdot \frac{8 \cdot l}{\Pi^2 \cdot d^5 \cdot g} \cdot Q^2$$

hr	=	head-loss pressure
l	=	length of the pipe
v	=	average velocity
A	=	cross section of the pipe
Q	=	discharge
g	=	acceleration

The factor lambda (λ) is dimensionless and depends on the velocity, pressure, temperature and viscosity of water. It is usually in the range of the transition zone and the ruff zone, which is described by the equation of Prandel-Colebrook:

$$\frac{1}{\sqrt{\lambda}} = -2 \cdot \lg\left(\frac{2,51}{Re \cdot \sqrt{\lambda}} + \frac{k/d}{3,71}\right)$$

the Reynold factor (Re) is defined as:

$$Re = -\frac{v \cdot d}{\vartheta} \qquad \vartheta \quad = \text{kinematic viscosity}$$

For the computer model the proposed pipe network including all knots, planned pipe lengths and elevations (ground levels) were used. The inflow and outflow as well as the pressure of the system at each knot had to be defined. The average velocity was determined as v = 0,8 - 1 m/s, to minimize the pressure losses and the pipe diameters. At the pumping main between the Water Works and the city network the velocity of the water was determined to be v = 2 - 3 m/s. On the basis of these data the pressure and pipe diameter were optimized.

The results show that pipe diameters of 40 mm to 500 mm have to be used for the pipe network. At each point of distribution and crossing, the pipe network requires a complete set of valves so it is possible to close down each individual sector of the pipeline in case of damage or repair without interrupting the water supply to the respective area.

The capacity of the network has been dimensioned considering the requirement of water for fire fighting purpose. Fire hydrants would have to be placed at a distance of 120 m. At dead ends, such as at the line to the Matrimandir, flushing hydrants have to be provided. The water pressure in the city pipe network at ground level varies from 4,15 bar up to 6,08 bar (see Annex 1.8 and drawing 42.01/1.2.1, 1.2.2).

The dimension of the Supply Main from the Water Works to the city has been estimated to be

DN 500 mm

Table 4.17: Required Pipe Diameter of the City Drinking Water Supply Network

Pipe Diameter in mm	DN 80	DN 100	DN 125	DN 150	DN 200	DN 250	DN 300	DN 400	DN 500	Total
Material	HDPE	HDPE	HDPE	HDPE	Cast Iron	Cast Iron	Cast Iron	Cast Iron	Cast Iron	
Length of Pipe in m	20.676	3.466	2.259	3.951	1.639	236	630	2.912	2.003	**37.772**

4.6 Irrigation Water Supply Network

4.6.1 Irrigation Water Supply

The water demand for irrigation has to been estimated (see 4.1.4) in a average year to be

$$Q_{Id} = 4{,}504{,}472 \text{ m}^3/\text{a}$$

This water requirement can be supplied by treated wastewater up to

$$Q_{WW} = 3{,}368{,}250 \text{ m}^3/\text{a}$$

and from groundwater sources

$$Q_{GW} = 1{,}136{,}222 \text{ m}^3/\text{a}$$

In a dry year the water demand for irrigation is estimated to be

$$Q_{Id} = 7{,}906{,}406 \text{ m}^3/\text{a}$$

This water requirement can be supplied by treated wastewater up to

$$Q_{WW} = 3{,}368{,}250 \text{ m}^3/\text{a}$$

and from groundwater sources

$$Q_{GW} = 4{,}538{,}156 \text{ m}^3/\text{a}$$

The water demand will be met in the city through a water supply network which runs parallel to the drinking water network. Every consumer in the city will be reached through this pipe system. This system will be fed by booster pumps from three intakes, from the Wastewater Treatment Plants in the West and the East of the township, as well as from the Water Works, where raw groundwater can be fed into the system. In a dry year the water demand in the GreenBelt cannot be met through the system. Individual tube wells have to be used to supply the peak flow.

4.6.2 Dimension of the Irrigation Water Supply Network

The Network has three intakes, from the Wastewater Treatment Plant West (WWTP-West), the Wastewater Treatment Plant East (WWTP- East), and from the Water Works.
From the (WWTP-West) the intake is proposed to be

daily flow	Q_{IW}	=	7510 m³/d
average flow	q_{IWmin}	=	87 l/s
peak flow	q_{Iwmax}	=	174 l/s

From the (WWTP-East) the intake is proposed to be

daily flow	Q_{IW}	=	2540 m³/d
average flow	q_{IWmin}	=	29 l/s
peak flow	q_{Iwmax}	=	58 l/s

From the groundwater source, the Water Works, the intake is proposed to be

max. daily flow	Q_{IW}	=	10,020 m³/d
peak flow	q_{IW}	=	0 to 116 l/s

The irrigation water distribution network is dimensioned to supply:

q_{max}	=	232 l/s
Q_{Id}	=	12,340 m³/d
Q_{Ia}	=	4,504,000 m³/a

For the dimensioning of the pipe network the computer programme from the company Barthauer based on the Hardy Cross Calculation has been used.

The friction losses have been calculated using the equation of Darcy-Weissbach:

$$hr = \lambda \cdot \left(\frac{l}{d} \cdot \frac{v^2}{2 \cdot g}\right)$$

with $v = \cdot \frac{Q}{A}$

The pressure losses can be derived from:

$$hr = \lambda \cdot \frac{8 \cdot l}{\Pi^2 \cdot d^5 \cdot g} \cdot Q^2$$

hr	=	head-loss pressure
l	=	length of the pipe
v	=	average velocity
A	=	cross section of the pipe
Q	=	discharge
g	=	acceleration

The factor lambda (λ) is dimensionless and depends on the velocity, pressure, temperature and viscosity of water. It is usually in the range of the transition zone and the ruff zone, which is described by the equation of Prandel-Colebrook:

$$\frac{1}{\sqrt{\lambda}} = -2 \cdot \lg\left(\frac{2{,}51}{Re \cdot \sqrt{\lambda}} + \frac{k/d}{3{,}71}\right)$$

the Reynold factor (Re) is defined as:

$$Re = -\frac{v \cdot d}{\vartheta} \qquad \vartheta$$ = kinematic viscosity

For the computer model the proposed pipe network including all knots, planned pipe lengths and elevations (ground levels) were used. The inflow and outflow as well as the pressure of the system at each knot had to be defined. The average velocity was determined as v = 0,8 – 1 m/s, to minimize the pressure losses and the pipe diameters. At the pumping main between the water works and the city network the velocity of the water was determined to be v = 2 - 3 m/s. On the basis of these data the pressure and the pipe diameter were optimized.

The results show that pipe diameters of 40 mm to 500 mm have to be used for the pipe network. At each point of distribution and crossing, the pipe network requires a complete set of valves so that it is possible to close down each individual sector of the pipeline in case of damage or repair without interrupting the water supply to the respective area.

The capacity of the network has been dimensioned considering the requirement of water for fire fighting purposes. Fire hydrants would have to be placed at a distance of 120 m. At dead ends, such as at the line to the Matrimandir, flushing hydrants have to be provided. The water pressure in the city pipe network at ground level varies from 3,5 bar up to 6,1 bar (see Annex 1.9 and drawing 42.01/1.2.3).

Table 4.18 Required Pipe Diameter of the City Irrigation Water Supply Network

Pipe Diameter in mm	DN 80	DN 100	DN 125	DN 150	DN 200	DN 250	DN 300	DN 400	DN 500	Total
Material	HDPE	HDPE	HDPE	HDPE	Cast Iron	Cast Iron	Cast Iron	Cast Iron	Cast Iron	
Length of Pipe in m	717	6.231	3.206	3.079	6.402	3.587	2.493	259	0	**25.974**

4.7 Estimated Costs

4.7.1 Estimated Costs for the Water Supply System

The estimated costs are based generally on unit prices available from India. In case these prices are not available, unit prices from Germany have been used. The following exchange rate has been used

1 Euro = 50 RS.

The costs for the construction of the drinking water extraction and for the Water Works have been estimated to be

103,111,000 RS.

The costs for the construction of the drinking water supply network have been estimated to be 91,869,000 RS.

The costs for the construction of irrigation water supply network have been estimated to be 92,318,000 RS.

The costs for the construction of 100 additional wells for the supply of irrigation water in dry years have been estimated to be

36,320,000 RS.

The total costs for the construction of the drinking water and irrigation water supply system have been estimated to be

323,618,000 RS.

Detailed cost estimates are presented in table 4.19 and Annex 1.10, 1.11, 1.12.

4.7.2 Estimated Costs for Operation and Maintenance of the Water Supply System

The annual costs for operation and maintenance of the water supply system have been estimated to be 14,053,300 RS/a.

The annual costs for the operation and maintenance of the irrigation water supply system have been estimated to be

16,273,700 RS/a.

In an average year the operation and maintenance of up to 100 wells for additional groundwater extraction have been estimated to be

2,106,100 RS/a.

These costs have to be included and the total maintenance costs for the irrigation water supply system are

18,379,800 RS/a.

The detailed cost estimates are presented in table 4.20.

4.7.3 Estimated Water Price

The costs for 1 cubic metre of drinking water has been estimated to be

32,83 RS/m³.

The costs of 1 cubic metre of process water from the cisterns in the households have been estimated to be

85,20 RS/m³.

The combined costs for one cubic metre of domestic water supply have been estimated to be

50,29 RS/m³.

The costs for 1 cubic metre of irrigation water has been estimated to be

3,61 RS/m³.

The detailed cost estimates are presented in table 4.21.

4.7.4 Estimated costs for the entire Water Management Scheme

The total costs for the entire water management scheme have been estimated to be

4,793,930,000 RS.

The detailed cost estimate are presented in table 4.19.

Table 4.19 Summary Estimate of Construction Costs

NO.	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	Total Costs [RS]	Total Costs * [€]
1	**DRINKING WATER SUPPLY**		
1.1	CONSTRUCTION OF DRINKING WATER EXTRACTION AND TREATMENT	103,111,000	2,062,000
1.2	CONSTRUCTION OF PIPE NETWORK FOR DRINKING WATER SUPPLY	91,869,000	1,837,000
1.3	CONSTRUCTION OF PIPE NETWORK FOR PROCESS AND IRRIGATION WATER SUPPLY	131,426,000	2,629,000
1	**SUBTOTAL**	**326,406,000**	**6,528,000**
2	**STORMWATER MANAGEMENT**		
2.1	CONSTRUCTION OF THE DOMESTIC CISTERN SYSTEM	2,315,270,000	46,305,000
2.2	CONSTRUCTION OF THE STORMWATER DRAINAGE SYSTEM	59,267,000	1,185,000
2.3	CONSTRUCTION OF THE STORMWATER RUNOFF SEDIMENTATION BASIN AND STORAGE TANKS IN THE GREEN BELT	849,560,000	16,991,000
2.4	CONSTRUCTION OF THE RAINWATER FILTRATION AND CONVEYANCE FROM THE GREEN BELT TO THE CITY	82,180,000	1,644,000
2.5	CONSTRUCTION OF THE RAINWATER RECIRCULATION AND FILTRATION IN PUBLIC PARKS	99,058,000	1,981,000
2.6	CONSTRUCTION OF THE MATRIMANDIR LAKE AND THE GROUNDWATER RECHARGE	394,907,000	7,898,000
2	**SUBTOTAL**	**3,800,242,000**	**76,005,000**
3	**WASTEWATER MANAGEMENT**		
3.1	CONSTRUCTION OF THE AUROVILLE SEWER NETWORK	189,552,000	3,791,000
3.2	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT EAST	121,215,000	2,424,000
3.3	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT WEST	356,515,000	7,130,000
3	**SUBTOTAL**	**667,282,000**	**13,346,000**
	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	**4,793,930,000**	**95,879,000**

* ... exchange rate 1€ = 50 RS

Table 4.20 Cost of Operation and Maintenance of the Water Supply System

No.	Description	Installed Capacity (Electrical Power)	Working Hours per Day (Electrical Power)	Working Days per Year	Annual Costs of Electrical Power Requirement *	Costs for Operation & Maintenance	Total Annual Costs	Total Annual Costs
	[-]	[kW]	[h/d]	[d/a]	[RS/a]	[RS/a]	[RS/a]	[Rs/a]
Total Annual Costs of Operation and Maintenance for the Water Supply System								
1	**Wells (Pumps)**	***150***	***24***	***365***	***6,570,000***	***908,000***	***7,47,000***	***149,560***
2	**Rapid Sand Filter**	***58***	***0,25***	***122***	***8,872***	***490,000***	***498,900***	***9,978***
3	**Sedimentation Basin**	***0,0***	***0,0***	***0,0***	***0,0***	***20,000***	***20,000***	***400***
4	**Drinking Water Storage Tank**	***0,0***	***0,0***	***0,0***	***0,0***	***100,000***	***100,000***	***2,000***
5	**Chlorination and Booster Pumps**							
5.1	*chlorination (max 320 l/s)*	*0,3*	*12*	*365*	*6,570*	*1,950*	*17,500*	*350*
5.2	*booster pumps (320 l/s, 9 bar)*	*265,9*	*12*	*365*	*5,822,619*	*72,000*	*5,894,600*	*117,892*
5	**Chlorination and Booster Pumps**				***5,829,189***	***82,950***	***5,912,100***	***118,242***
6	**Drinking Water Supply Network**				***5,829,189***	***82,950***	***5,912,100***	***118,242***
Total annual Costs of Operation and Maintenance for the Water Supply System							**14,009,000**	**280,180**
Total annual Costs of Operation and Maintenance for the Irrigation Water Supply System **								
7	**Booster Pumps for Irrigation**							
7.1	*Booster Pumps (175 l/s, 6,5 bar)*	*125,0*	*12*	*215*	*1,612,500*	*37,500*	*1,65,000*	*33,000*
7.2	*Booster Pumps (60 l/s, 5 bar)*	*42,9*	*12*	*215*	*552,857*	*17,500*	*570,400*	*11,408*
7	**Booster Pumps for Irrigation**				***2,165,357***	**55,000**	***2,22,400***	***44,408***
8	**Irrigation Water Supply Network**				***2,165,357***	**55,000**	***2,220,400***	***44,408***
Total annual Costs of Operation and Maintenance for the Irrigation Water Supply System **							**2,220,400**	**44,408**
TOTAL ANNUAL COSTS OF OPERATION AND MAINTENANCE OF THE WATER SUPPLY SYSTEM							**16,229,400**	**324,588**
Total Annual Costs of Operation and Maintenance for the Complementary Irrigation Water Wells **								
9	**Wells (Pumps)**	***150***	***7,43***	***215***	***1,198,088***	***908,000***	***2,106,100***	***42,122***
Total Annual Costs of Operation and Maintenance for the Complementary Irrigation Water Wells **							**2,106,100**	**42,122**

*... rate per kWh is RS 5,0
**... Irrigation Demand - average year

Table 4.21 Estimate of the Costs for Water Supply

	Description	Construction Costs	Costs for M&O	Drinking Water[1)] Costs / m^3		Process Water[2)] Costs / m^3		Drinking and Process Water[3)] Costs / m^3		Irrigation Water[4)] Costs / m^3		Wastewater[5)] Costs / m^3	
No.	Costs for the Sanitary Infrastructure of Auroville	[RS]	[RS/a]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]
1	**WATER SUPPLY ***												
1.1	Drinking Water Extraction and Treatment	103,111,000	14,009,000	4,38	0,09			2,92	0,06				
1.2	Pipe Network for Drinking Water Supply	91,869,000	5,912,100	2,11	0,04			1,41	0,03				
1.2	Pipe Network for Process and Irrigation Water Supply	131,426,000	4,440,800							3,61	0,07		
1	**SUBTOTAL**	**326,406,000**	**24,361,900**	**6,49**	**0,1**	**0,0**	**0,0**	**4,33**	**0,09**	**3,61**	**0,07**	**0,00**	**0,00**
2	**STORMWATER MANAGEMENT ****												
2.1	Domestic Cistern System	2,315,270,000	57,881,750			85,2	1,70	28,40	0,57				
2.2	Stormwater Drainage System	59,267,000	4,523,893	1,39	0,03			0,93	0,02				
2.3	Stormwater Runoff Sedimentation Basins and Storage Tanks in the GreenBelt	849,560,000	33,982,400	11,58	0,23			7,72	0,15				
2.4	Rainwater Filtration and Conveyance from the GreenBelt to the City	82,180,000	6,163,500	1,90	0,04			1,27	0,03				
2.5	Rainwater Circulation and Filtration in Public Parks	99,058,000	7,429,350	2,30	0,05			1,53	0,03				
2.6	Matrimandir Lake and Groundwater Recharge	394,907,000	29,618,025	9,15	0,18			6,10	0,12				
2	**SUBTOTAL**	**3,800,242,000**	**139,598,918**	**26,33**	**0,53**	**85,2**	**1,70**	**45,95**	**0,92**	**0,00**	**0,00**	**0,00**	**0,00**
3	**WASTEWATER MANAGEMENT**												
3.1	Sewer Lines**	189,552,000	636,173									0,69	0,01
3.2	Wastewater Treatment Plant East*	121,215,000	9,091,125									3,14	0,06
3.3	Wastewater Treatment Plant West*	356,515,000	26.738,625									9,23	0,18
3	**SUBTOTAL**	**667,282,000**	**36,465,923**	**0,00**	**0,00**	**0,0**	**0,00**	**52,05**	**1,04**	**0,00**	**0,00**	**13,06**	**0,26**
	TOTAL COSTS FOR WATER SUPPLY	**4,793,930,000**	**200,426,741**	**32,82**	**0,66**	**85,2**	**1,70**	**102,33**	**2,05**	**3,61**	**0,07**	**13,06**	**0,26**

*... Time of Depreciation 50 years

**... Time of Depreciation 100 years

1)... with 3.668.250,00 m^3/a

2)... with 1.222.750,00 m^3/a

3)... with 2.445.500,00 m^3/a for Drinking Water and 1222750 m^3/a for Process Water

4)... with 1.956.400,00 m^3/a

5)... with 3.668.250,00 m^3/a

5 PRE - FEASIBILITY STUDY FOR THE STORMWATER MANAGEMENT OF THE CITY OF AUROVILLE

5.1 Introduction

As part of the integrated approach to water resources management in the city of Auroville the setting up of a Rainwater Harvesting System (RWHS) for utilization of available rainwater is recommended (see drawing 42.02/1.3.7).

5.2 Existing Stormwater Management

In the past, the plateau of Auroville had no vegetation cover, almost the entire stormwater runoff was drained through canyons down the hill into the sea. Only a small fraction of rainfall infiltrated the ground.

At present almost the entire Auroville area is “bunded”, afforested and covered with vegetation in order to prevent erosion and runoff. Stormwater runoff is minimized and most of the rainfall infiltrates underground.

Several check dams have been built in the canyons to retain the eroded soil, to store stormwater runoff and to infiltrate stored water for groundwater recharge.
At present, very few buildings provide facilities for stormwater harvesting and its usage. Rainwater when it is harvested is only used for gardening.

5.3 The Proposed Stormwater Management System

5.3.1 Objectives

The primary objective of the storm-water harvesting for rainwater utilization is to develop a water resource management system to ensure and secure the availability of freshwater so that the water demand in Auroville is sustainably met. Over the coming decades, the management of water resources will become one of the most important issues across industrialized nations as water availability and quality are likely to decrease. Given the already existing water problems encountered in Auroville, it is imperative that all sources of freshwater be considered and if possible tapped to ensure a safe and secure supply.

Given the scarcity of freshwater in Auroville and the surrounding regions, it is proposed to obtain additional sources of freshwater by harvesting all the rainwater that falls on the paved and unpaved surfaces. This is described in detail below. Through the harvesting of rainwater for direct use and groundwater recharge an alternate and reliable source of freshwater is made available.

In addition to solving the problems of freshwater supply, this proposal also seeks to implement systems and technologies that are appropriate for the prevailing physical, environmental, social and economic conditions the project site. Thus concepts for systems and technologies that adhere to the principles of resource optimization and sustainable development have been developed and proposed here. In particular it has been ensured that the proposed systems and technologies meet the following criteria. (see drawing 42.02/1.1.1).

- Minimum dependency on complex infra-structure services,
- High self-sufficiency in respect to operation and maintenance of systems,
- Low vulnerability to destruction,
- Can accommodate significant variations in hydraulic and pollution loads without significant loss of efficiency,
- Can handle a large variety of pollutants present in today’s stormwater runoff,
- High efficiency in treatment of stormwater and removal of pathogens,
- No , or limited use of, mechanical parts (except for the minimum use of pumps for the required lifting of stormwater),
- Use of simple hardware,
- Minimized inputs of energy,
- No use of chemicals for the treatment process,
- No requirement of skilled manpower,
- Low long-term capital, operating and maintenance costs,
- Applicable at any site and scale.

- Allows phasing of systems,
- Can be easily and cost-effectively expanded to accommodate increased loads,
- Simple construction,
- Use of appropriate and suitable materials,
- Use of indigenous materials and building technologies to the maximum extent,
- Allows re-cycling and safe use of rainwater,
- Allows the full infiltration of the stormwater for the recharge of the first aquifer,
- Prevents environmental pollution problems, in particular pollution of air, water and soil,
- Ensures environmental protection,
- Enhances or maintains the quality of the surrounding environment.

5.3.2 Description of Project Components

5.3.2.1 Roof top stormwater runoff

For the harvesting and utilization of rainwater in Auroville, a system that allows the harvesting and direct utilization of rainwater at each building has been proposed. Such a system would be able to:

- Handle polluted rainwater runoff from all roof tops in the city of Auroville,
- Store in an optimal manner the volume of rainwater that is required for direct utilization, and improve and maintain its quality while it is being stored,
- Provide water of quality suitable for use in toilet flushing, washing machine etc,
- Infiltrate the excess rainwater into the soil for groundwater recharge in a manner that reliably ensures the prevention of groundwater pollution.

The proposed rainwater harvesting system at all private and public premises shall consist of the following principal components:

Figure 5.1 Scheme of Rainwater Harvesting and Re-use for the City of Auroville

1. Storm water runoff collection and drainage system,
2. Storm water runoff pre-filtration system,
3. Storm water runoff treatment system,
4. Storm water runoff storage system,
5. Post-storage rainwater filtration system,
6. Infiltration system for groundwater recharge of excess rainwater,
7. Supply and distribution system for utilization of harvested rainwater.

This is illustrated in figure 5.1 and drawing 42.02/1.1.2.

In enclosed private residential or public premises within the city of Auroville stormwater runoff from paved ground surfaces will be drained into open green spaces and infiltrated into the ground. Stormwater runoff from open green spaces will be prevented by contour bunding. No stormwater will be drained from the premises.

5.3.2.2 Surface and public roads stormwater runoff

The stormwater runoff from all the roads, public spaces, public parks and the GreenBelt will be drained and harvested.

For the harvesting and utilization of surface and road stormwater runoff from public spaces, a system that allows the harvesting and direct utilization of rainwater has been proposed. Such a system would be able to:

- Handle polluted rainwater runoff from all the paved and unpaved surfaces in the city of Auroville and the GreenBelt,
- Store in an optimal manner the volume of stormwater runoff that is required for re-use and groundwater recharge, and improve and maintain its quality while it is being stored,
- Provide water of quality suitable for groundwater recharge,
- Infiltrate the purified stormwater runoff into the soil for groundwater recharge in a manner that reliably ensure non- pollution of the aquifer.

The proposed stormwater harvesting system shall consist of the following principal components:

1. Stormwater runoff collection and drainage system,
2. Stormwater runoff pre-treatment system,
3. Stormwater runoff storage system,
4. Post-storage re-circulation system
5. Post-storage rainwater filtration system,
6. Stormwater runoff treatment system,
7. Infiltration system for groundwater recharge,

This is illustrated in figure 5.1 and drawing. 42.02/1.1.2.

5.3.3 Description of the Drainage Area

5.3.3.1 Location

The drainage area comprises the built-up area of the city of Auroville, which is a perfect circular area with a surface area of 4,9 km² and a diameter of 2,5 km on the top of a hill with the highest point at its centre.

The built-up area is split into 4 zones by four public parks. The four zones are the Residential Zone, the International Zone, the Industrial Zone and the Cultural Zone.

The outer limit of the city area is determined by the Outer Ring-Road at a radius of 1,250 m from the centre. The Crown Road (the Inner Ring Road) located at a radius of 700 m from the centre separates the 4 zones from the city centre, which contains the Peace Area with the Matrimandir in its middle. Each zone is framed by a Radial Road that acts as a main access road.

5.3.3.2 Topography

The city centre is located just next to the top of the hill that has an elevation of 54 m above mean sea level (MSL).

From the centre, the surface slopes down to the elevation of 43 m above MSL in the North, to 34 m above MSL in the West, to 46 m above MSL in the East and 50 m to the South. There is a major watershed from the North-East to South-West splitting the city area into two major catchment areas. The slope of the ground ranges from 0,2 % to 0,7 % which allows the area to be drained comfortably by gravity.

5.3.3.3 Land Use

According to the Master Plan, the land use of the Auroville area, the five zones and the GreenBelt is proposed as indicated in table 5.1 (see drawing 42.01/1.1.1).

Table 5.1: Proposed Land Use Zones – 2025 (City Area / Developed Area)

Use Zones	Area in ha	%	Principal Uses
Peace Area	**28.00**	**5.70**	Matrimandir, Lake, Gardens
1. Residential Zone	**173.00**	**35.20**	
Primary Residential	160.000	32.60	Residential houses, apartments in five sectors and different densities and basic community facilities
Crown	23.00	2.60	Shopping, Utilities, Communication, recreation and community facilities of higher orders, supporting residential use.
2. International Zone	**68.00**	**13.90**	
Pavilions	63.50	12.90	National and International Pavilions, conference and exhibitions halls
Crowns	8.50	1.00	Utilities, Communication, Shops and other Common Facilities related to the main activity in the International Zone, Including Housing and Staff Quarters
3. Industrial Zone	**126.00**	**25.70**	
Economic	94.50	19.30	Non-polluting Manufacturing units, including Cottage Industries
Crown	5.50	1.70	Hotels, Dormitories, Guest Houses and Supporting Facilities for the main activity in the zone.
Administration	7.00	1.40	Town Hall, City Administration Office and Housing
Vocational Training	16.00	3.30	Vocational Training Centres, Research Institutions including Laboratories
4. Cultural Zone	**96.00**	**19.50**	
Major cultural	91.00	18.50	Educational institutions, University, Sports Centres and Staff Quarters.
Crown	5.00	1.00	Shopping, Utility, Communication and Recreation Centres and related facilities supporting Cultural Activities in the zone including Housing.
Total	**491.00**	**100.00**	

Table 5.2: Proposed Land Use in the Green Belt – 2025

		Area in ha	%	Principal Uses
	Built (Existing settlements to be retained)	156	10.50	Auroville Communities and Village Residential Areas, Service Nodes and Utilities and Main Access Roads.
	Unbuilt	1316	89.50	Farming and Forest type uses and Recreation, Bird and Wild Life
Total		**1472**	**100.00**	

The land use within the drainage area of the city according to the Master Plan is defined in Table 5.3.

Table 5.3: Detailed Land Use in City Area - 2025

Use	Extent in ha	%	Remarks
1. Residential	121	24.64	Residentia Zone 80 %,Other zones 20%
2. Commercial	20	4.10	Mostly in Crown Area connecting in zones
3. Industrial	56	11.40	Industrial Zone / Manufacturing Units
4. Public & Semi-public	159	32.38	
a. Matrimandir	28	5.70	Peace Area
b. Pavilions	38	7.73	International Zone
c. Educational & Cultural	73	14.86	Cultural and Residential
d. Administration, utilities & other uses	20	4.07	Industrial and other zones
5. Open space & recreation	46	9.36	To be provided in all zones
6. Transport & communication	89	18.12	To serve all zones
Total	**491**	**100.00**	

5.3.3.4 Road Network

According to the Master Plan the physical infrastructure is planned as follows:

Road network: The road network, consisting of four types of roads, is planned to meet the future requirement of traffic and functioning of the Township. The proposed road network is shown in the proposed land use plan as well as in Drawing 7 on hierarchy of roads. Road section is shown in Drawing 8. The four types of roads and access ways in order of hierarchy are as under:

Access Roads to Auroville: Four principal accesses are proposed. Two from the Tindivanam-Pondicherry Road, connecting the Industrial Zone and the International Zone. The other two accesses are from the East Coast Road (ECR), which would link the Residential Zone and the Cultural Zone. Thus each zone will have an independent access from state / national highways. These roads will provide links to the Outer Ring Road of the City.

There would be bypass links where the existing narrow roads pass through village settlements. The right of way of these roads is suggested to be 30 meters.

City Ring Roads: Two ring roads are proposed within the City area, one circumscribing the four main use zones and the other adjoining the utility zone, which is designed as the Crown road. The right of the way of these is also suggested to be 30 meters. These two ring roads will help in distributing the traffic to the different zones.

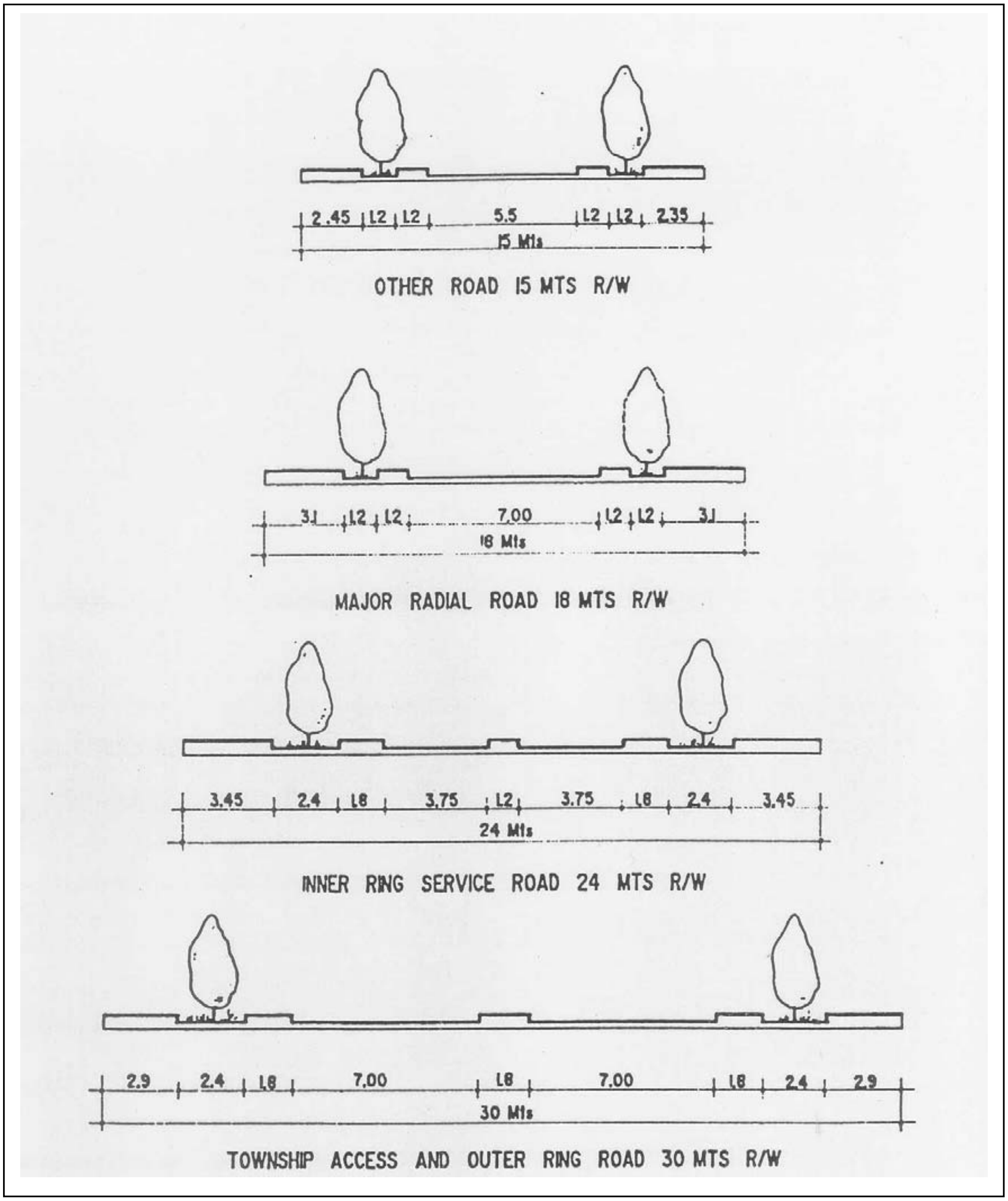
The entire City area has been envisaged as a “non-polluting vehicular zone”. Accordingly, the ring road circumscribing the City Area will be used increasingly by non-polluting vehicles.

Internal Distribution Roads: The internal distribution consists of vehicular roads as well as pedestrian and cycle paths. The rights of way of vehicular roads would vary between 18 and 24m depending upon the function.

Service Nodes: Two kinds of service nodes are proposed. The service nodes provided in the Green Belt are proposed at the intersection of the four main access roads linking the township and the City area. The first one would be called Primary Node and the latter the Secondary Service Node, as indicated in the schematic layout of Service Nodes. These Service Nodes will provide adequate parking and trans-shipment space for changing over to “non-polluting” transport before entering the City. These Service Nodes will also offer other facilities for providing a convenient interface with neighbouring village settlements.

In addition to the main categories of roads discussed above, two bypass roads are also suggested, one in the north and the other in the south of the township to facilitate diversion of traffic not destined for Auroville.

Figure 5.2: Road Sections



In the Auroville Mobility Concept of Planungsbüro Billinger, Stuttgart 2001, a modification of the traffic and road concept was proposed as shown in Figs. 5.3, 5.4 and 5.5.

The aim of the Mobility Concept was "...to work out the mobility parameters of the Master Plan with more details. Based on the Master Plan's general considerations on traffic, especially the aim of giving preference to non-polluting movement, a network of roads and pathways has been proposed. A shuttle bus system is recommended to complete the network, connected to the Service Nodes specially developed for Auroville. In conclusion, some recommendations have been given as to how a motor-free city can be realized in carefully chosen steps."

Table 5.4: Auroville Town Plan – Basic Distances

Location	**Length and Distances**		**Walking Time***
		Metres	Minutes
Crown Road	Radius Diameter Circumference	700 1400 4400	**10** **20** **63**
Outer Ring Road	Radius Diameter Circumference	1250 2500 8000	**18** **36** **114**
Green Belt limit	Radius Diameter Circumference	2500 5000 16000	**36** **71** **228**
Peace Area – Crown Road Crown Road – Outer Ring Road Outer Ring Road – Green Belt limit Outer Ring Road Diagonal Green Belt limit Diagonal		350 550 1280 2800 5000	**5** **8** **18** **36** **71**

*In meters and minutes walking time / Speed of walking: 70 m per min.

Figure 5.3: Auroville Township dimensions (Source: Auroville Mobility Concept)

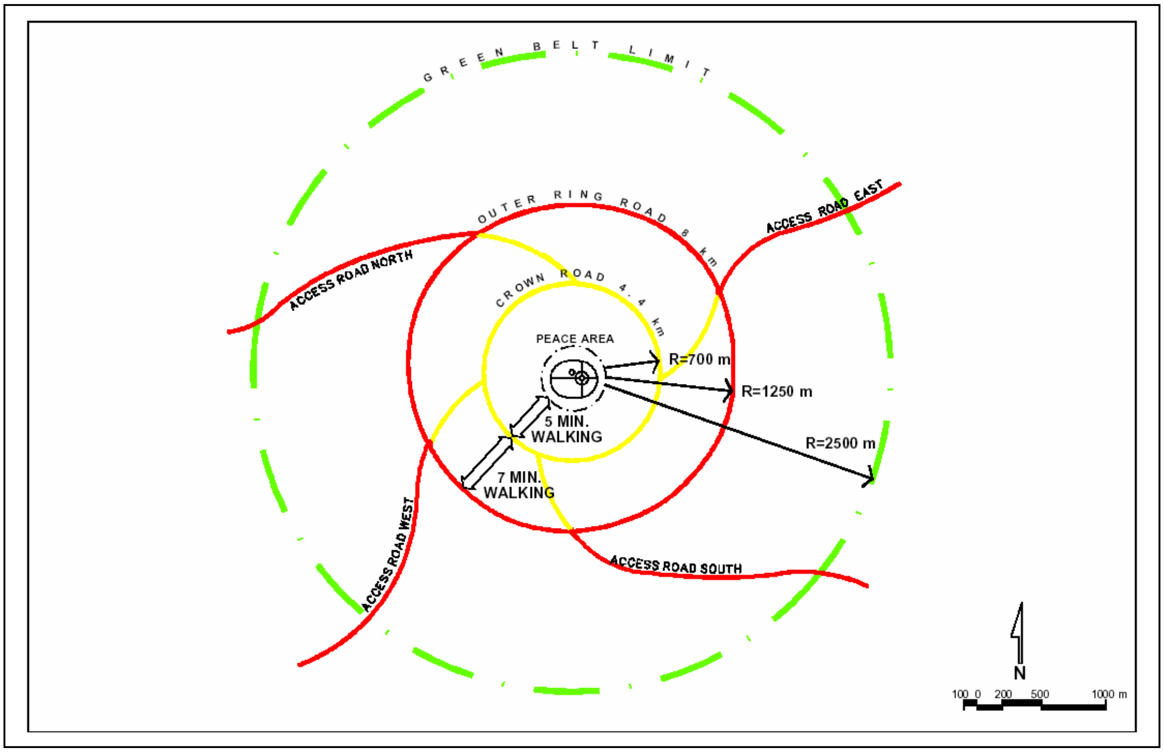


Figure 5.4: General Mobility Pattern (Source: Auroville Mobility Concept

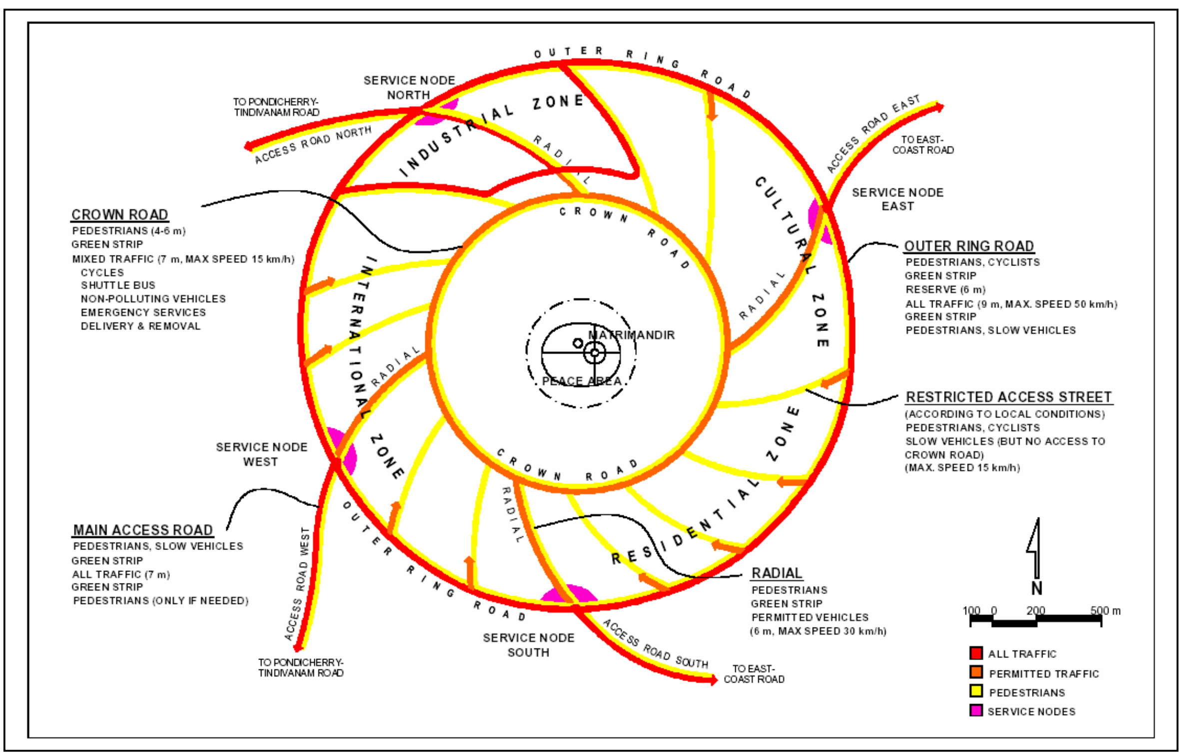
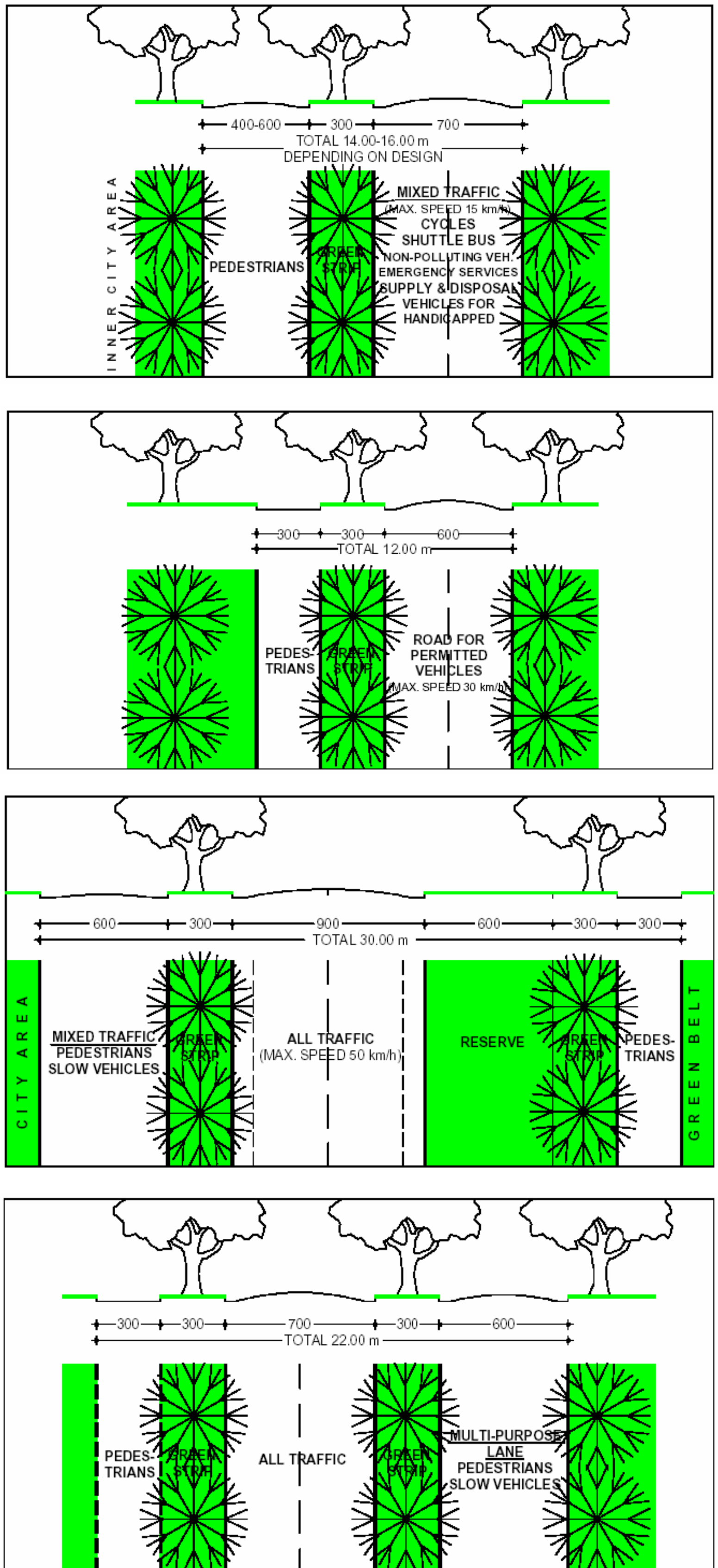


Figure 5.5: Section of Roads (Source: Auroville Mobility Concept)



5.3.3.5 Drainage Areas

Auroville covers an area of 1,963 ha out of which the city area covers 491 ha. The city area is split into five zones according to the Master Plan as shown in table 5.5.

Table 5.5: Drainage Area of Auroville City

Location	Drainage Area
Residential Zone	160 ha
International Zone	63 ha
Industrial Zone	95 ha
Cultural Zone	91 ha
City Centre	82 ha
Total of drainage city area	491 ha
The GreenBelt covers an area of	1,472 ha
The total drainage area of Auroville covers a surface of	1,963 ha

The city area drains into the GreenBelt from 3 catchment areas, while the GreenBelt drains into the existing canyons from 7 catchment areas. There exists one major watershed, running from northeast to southwest through the centre of the city. The drainage area has been differentiated into 6 different types of area according to the degree of sealing and the use of the areas:

- impervious areas (path network areas, traffic zones, roads)
- private green areas (garden area)
- public green space
- GreenBelt
- major water body area
- roof area.

The surface areas of the roads have been based on the cross sections given in the Master Plan. The area covered by internal roads within the built-up area have been estimated as shown in Table 5.6:

Table 5.6: Stormwater Drainage Areas

Catchment Area	Impervious Area	Private Green Area	Public Green Area	Green Belt	Lake Area	Roof Area	Total Drainage Area
	[ha]	[ha]	[ha]	[ha]	[ha]	[ha]	[ha]
GB-I	36			84	0	0	120
GB-II	24	49	23	228	2	0	326
GB-III	60	15	6	198	4	0	283
GB-IV	123	111	75	235	43	0	587
GB-V	24			76	0	0	100
GB-VI	24			127	9	0	160
GB-VII	0			258	2	0	260
Cistern						107	107
Matrimandir Lake					18	0	18
Total	290	175	104	1,206	**78**	**78**	1,960

(see Annex 2.1, 2.2)

5.4 Proposed Stormwater Drainage System

5.4.1. Methods of Drainage

The proposed concept for utilization of stormwater runoff and the re-use of sewage requires two separate drainage systems. It is proposed to drain stormwater in open collectors. The sewage has to be drained in a closed system, using stoneware pipes.
In locations where the use of an open channel is not feasible, reinforced concrete pipes have to be used for the conveyance of the stormwater.

The topography of the city of Auroville shows one major watershed and permits the design of the drainage system entirely based on gravity flow using only two major catchment areas.

The largest catchment area drains by gravity inland towards the northwest, and the smaller catchment area drains to the southeast towards the Bay of Bengal. All the drains in the entire territory will be integrated into the road network and will connect to secondary drains that will be located in the Green Corridors, linking the various sectors and zones. The secondary drains will connect to the main drains located in the park areas, draining the parks as well as the zones into the GreenBelt. The main drains will connect to natural drains, such as the canyons in the GreenBelt which lead to the city limit. The city boundary will be protected by a boundary bund and a boundary drain (see drawing 42.02/1.3.6). The runoff in the GreenBelt that is not collected by the main drains or by the natural drains will be collected in the boundary drain. In the GreenBelt the northern watershed is separated into four drainage catchment areas, and the south western watershed into three drainage catchment areas.

The runoff from the main drains of the city area and the natural drains of the GreenBelt in each of the seven drainage catchment areas will be drained into existing or artificial ponds or storage tanks to be used to cover the water demand of the city (see drawing 42.02/1.2.1, 1.2.2).

5.4.2. Dimensioning of the Stormwater Drainage System

5.4.2.1 Rainfall

Rainfall data are available from the following organizations:

- Regional Weather Station, Pondicherry (1911-1971,1984-1991),
- Auroville - Certitude (1972-1983),
- Public Works Department, Pondicherry (1992-1995),
- Auroville-Aurodam (1996,1997),
- Auroville-Harvest (1998-2001).

All further calculations and simulations are based on the following rainfall data for a dry, average and wet year:

Table 5.7: Rainfall data

Month	Precipitation Average Year	Precipitation Wet Year	Precipitation Dry Year
	[mm]	[mm]	[mm]
January	36	88	25
February	18	48	18
March	19	0	0
April	22	47	0
May	45	1,098	38
June	45	67	17
July	68	28	26
August	118	157	56
September	145	139	5
October	263	367	202
November	350	234	25
December	162	331	214
Annual	**1,293**	**2,604**	**626**

For further data see Annex 2.2.

The runoff from the proposed surfaces has been calculated for a rainfall intensity of

$$r = 300\ l/s \cdot ha$$

The runoff factor has been estimated to be:

main roads	$\Psi'_{MR} = 0{,}8$
green space, parks	$\Psi'_{G} = 0{,}4$
internal roads	$\Psi'_{IR} = 0{,}8$

5.4.2.2 Hydraulic Calculation of the Stormwater Drains

Public roads and places will be drained in roadside drains or troughs. Parallel to the roads, in the road-side green verge, a lined ditch will be provided to receive at intervals of 50 m to 100 m the runoff from the paved part of the road. These lined ditches drain into open lined channels in the Green Corridors or parks. The ditches and channels will have a trapezoidal cross section and the side walls will slope of 1 : 2.

The bottom width and the depth of the ditches and channels will be adjusted to the variations of the ground level. The depths shall range from 0,4 m to 1,0 m. The width of the channel will range from 1,6 m to 6,5 m and the bottom width from 0 to 2,5 m.

The flow in the channel has been calculated according to the continuity equation:

$$Q = A \times v$$

The velocity v has been calculated according to the Manning-Strickler equation:

$$v = k_{ST} \times R^{2/3} \times I^{1/2}$$

with $R = U / A$

with the use of

Q	Discharge
k_{ST}	Roughness according to Manning-Strickler
A	area of the cross section of the flowing water
R	hydraulic radius
U	wetted perimeter
I	Gradient of the channel

For the trapezoidal cross section:

$$A = b \times h + m \times h^2$$

For the wetted Perimeter:

$$U = b + 2 \cdot h\sqrt{1+m}$$

with the use of:	b	bottom width of the channel
	h	depth of the channel
	m	side slope of the channel 1 : m

The discharge capacity of a trapezoidal channel can be calculated by the following equation:

$$Q = k_{ST} \cdot (b \cdot h + m \cdot h^2) \cdot \left[\frac{b \cdot h + m \cdot h^2}{b \cdot 2h \cdot \sqrt{1+m^2}} \right]^{2/3} \cdot I^{1/2}$$

The roughness of the channel lining has been calculated according to Manning-Strickler:

$$k_{ST} = 70$$

The discharge of the trapezoidal profiles used in the stormwater drainage can be obtained from Table 5.8. The dimensions of the profiles and the discharge can be obtained from the list of calculations of the entire drainage system in seven catchment areas (GB I to VII) from Annex 2.8 and 2.9. The drainage of the city area requires a total length of stormwater drains and channels of 38,735 m. The discharge of the main drains in each catchment area can be obtained from Table 5.9. The length of the trapezoidal drainage channels can be obtained from Table 5.10.

(see drawings 42.02/1.2.1, 1.2.2, 1.3.1, 1.3.2, 1.3.3, 1.3.4).

Table 5.8: Discharge for Drains with Trapezoidal Profile

Discharge Q_{max} [in m³] for Drains with Trapezoidal Profile - side slope 1:2 and full filling of the profile

Profile Type				l	k	j	i	h	g	f	e	d	c	b	a
Water depth	h	[m]		1	1	1	1	0.9	0.8	0.7	0.6	0.5	0.4	0.4	0.4
Roughness	k_{ST}	$[m^{1/3}/s]$		70	70	70	70	70	70	70	70	70	70	70	70
Drain bottom width	B	[m]		3	2.5	2	1	1	1	1	1	1	1	0.5	0
Drain width on top	BT	[m]		7.00	6.50	6.00	5.00	4.60	4.20	3.80	3.40	3.00	2.60	2.10	1.60
Q with Slope I = 1:	50		0.0200	37.87	33.27	28.73	19.89	15.75	12.17	9.13	6.59	4.52	2.89	1.92	1.01
Q with Slope I = 1:	100		0.0100	26.78	23.53	20.32	14.07	11.13	8.61	6.46	4.66	3.20	2.04	1.36	0.71
Q with Slope I = 1:	150		0.0067	21.86	19.21	16.59	11.49	9.09	7.03	5.27	3.81	2.61	1.67	1.11	0.58
Q with Slope I = 1:	200		0.0050	18.93	16.64	14.37	9.95	7.87	6.08	4.56	3.30	2.26	1.44	0.96	0.50
Q with Slope I = 1:	250		0.0040	16.93	14.88	12.85	8.90	7.04	5.44	4.08	2.95	2.02	1.29	0.86	0.45
Q with Slope I = 1:	300		0.0033	15.46	13.58	11.73	8.12	6.43	4.97	3.73	2.69	1.85	1.18	0.78	0.41
Q with Slope I = 1:	350		0.0029	14.31	12.57	10.86	7.52	5.95	4.60	3.45	2.49	1.71	1.09	0.72	0.38
Q with Slope I = 1:	400		0.0025	13.39	11.76	10.16	7.03	5.57	4.30	3.23	2.33	1.60	1.02	0.68	0.36
Q with Slope I = 1:	450		0.0022	12.62	11.09	9.58	6.63	5.25	4.06	3.04	2.20	1.51	0.96	0.64	0.34
Q with Slope I = 1:	500		0.0020	11.97	10.52	9.09	6.29	4.98	3.85	2.89	2.08	1.43	0.91	0.61	0.32
Q with Slope I = 1:	550		0.0018	11.42	10.03	8.66	6.00	4.75	3.67	2.75	1.99	1.36	0.87	0.58	0.30
Q with Slope I = 1:	600		0.0017	10.93	9.60	8.29	5.74	4.55	3.51	2.64	1.90	1.31	0.83	0.55	0.29
Q with Slope I = 1:	650		0.0015	10.50	9.23	7.97	5.52	4.37	3.38	2.53	1.83	1.25	0.80	0.53	0.28
Q with Slope I = 1:	700		0.0014	10.12	8.89	7.68	5.32	4.21	3.25	2.44	1.76	1.21	0.77	0.51	0.27
Q with Slope I = 1:	750		0.0013	9.78	8.59	7.42	5.14	4.07	3.14	2.36	1.70	1.17	0.75	0.49	0.26
Q with Slope I = 1:	800		0.0013	9.47	8.32	7.18	4.97	3.94	3.04	2.28	1.65	1.13	0.72	0.48	0.25
Q with Slope I = 1:	850		0.0012	9.18	8.07	6.97	4.82	3.82	2.95	2.21	1.60	1.10	0.70	0.46	0.24

Table 5.9: Discharge of the Main City Stormwater Drains

Catchment Area	Surface of the Catchment Area	Surface Area of the Storage Tanks	Main Drain Profile	Discharge
	[ha]	[ha]	H [m] / B [m]	[l/s]
GB-II, RW 3000	44	507,000	1,0 / 0,6	2,601
GB-II, RW 3900	96		1,0 / 0,8	5,928
GB-III, RW 3100	31	555,000	1,0 / 0,5	1,923
GB-IV, RW 1000	127	1,174,000	1,0 / 0,9	8,796
GB-IV, RW 2000	90		1,0 / 0,8	5,955
GB-IV, RW 4000	127		2,5 / 1,0	11,946
Total	**515**	**2,236,000**		**37,149**

Table 5.10: Length of the Stormwater Drains with Trapezoidal Profile

Profile			**l**	**k**	**j**	**i**	**h**	**g**	**f**	**e**	**d**	**c**	**b**	**a**	**Total Length**
Water Depth	h	m	1	1	1	1	0,9	0,8	0,7	0,6	0,5	0,4	0,4	0,4	
Drain Width on Basis	B	m	3	2,5	2	1	1	1	1	1	1	1	0,5	0	
Length	L	m	**0**	**547**	**351**	**134**	**113**	**1,563**	**871**	**1,036**	**3,276**	**3,377**	**9,795**	**17,673**	**38,735**

(see drawings 42.02/1.2.1 and 1.2.2).

5.5 Proposed Stormwater Management

5.5.1 Methods of Stormwater Management

The runoff from the main drains of the city area and the natural drains of the GreenBelt in each of the seven drainage catchment areas will be drained into existing or artificial ponds or storage tanks. All drains have to be protected against erosion and sealed against seepage. The velocity of the flow should prevent the sedimentation of grit and silt. The inflow to the tanks will be treated by passing it through a sedimentation basin for the removal of grit and floating material.
The tanks have to be sealed to prevent leakage, and deep to minimize evaporation losses.

From the tanks, the stormwater runoff has to be treated and pumped into a re-circulation system, located in the four parks of the city. This system consists of an interconnected series of springs, water courses and ponds.

After re-circulation the stormwater has to be further treated and pumped into the central rainwater lake surrounding the Matrimandir where it will undergo extensive biological and natural treatment make it suitable for groundwater recharge. From there the water is infiltrated into the first aquifer. All drains have to be protected against erosion and sealed against seepage. The velocity of the flow should prevent the sedimentation of grit and silt. (see drawing 42.02/1.2.3).

5.5.2 Stormwater Runoff Storage Tanks

5.5.2.1 Rooftop Runoff Storage Tanks

For the harvesting and utilization of rainwater in Auroville, a system that allows the harvesting and direct utilization of rainwater at each building has been proposed. The rainwater harvesting system at all private and public premises will consist of the following principal components:

1. Stormwater runoff collection and drainage system,

2. Stormwater runoff pre-filtration system,

3. Stormwater runoff treatment system,

4. Stormwater runoff storage system,

5. Post-storage rainwater filtration system,

6. Infiltration system for groundwater recharge of excess rainwater,

7. Supply and distribution system for utilization of harvested rainwater.

This is illustrated in figure 5.1, drawing 42.02/1.1.2 and Annex 2.7.1, 2 and 3.

The total rooftop area in Auroville has been estimated to be

$$A_R = 1{,}066.500\ m^2$$

The total annual runoff from this rooftop area has been estimated for a dry, a mean and a wet year at

$$Q_{dry} = 852{,}923\ m^3/a$$
$$Q_{mean} = 1.227{,}648\ m^3/a$$
$$Q_{wet} = 1.858{,}941\ m^3/a$$

The required storage volume for the rainwater harvesting has been estimated to be:

$$V_{dry} = 279{,}386\ m^3$$
$$V_{mean} = 469{,}977\ m^3$$
$$V_{wet} = 845{,}829\ m^3$$

For this storage volume a population of 50,000 can be supplied with harvested rainwater at a rate per capita and day of

$$q_{dry} = 46{,}7\ l/cap \cdot d$$
$$q_{mean} = 67{,}3\ l/cap \cdot d$$
$$q_{wet} = 101{,}9\ l/cap \cdot d$$

For the supply of 64,000 Population Equivalents (3,000 inhabitants in the GreenBelt have been subtracted) the supply rate per PE and day with harvested rainwater can be estimated to be:

Table 5.11/1: Rooftop Rainwater Harvesting

Selected Year	Area of Rooftops	Rooftop Runoff	Estimated Volume of Cisterns	Process Water Supply	Process Water Supply	Process Water Supply
	[m²]	[m³]	[m³]	[m³]	[l/PE d]	[l/capita d]
Dry Year	1.066.500	50.072	265.966	50.072	24	32
Average Year	1.066.500	103.408	448.235	103.408	49	67
Wet Year	1.066.500	208.287	834.371	208.287	99	134

5.5.2.2 Stormwater Runoff Storage Tanks

The stormwater runoff from the city and the GreenBelt has been estimated for a year with average precipitation, as well as for a dry and for a wet year as indicated in table 5.11/2.

Table 5.11/2: Annual Stormwater Runoff from the City and the GreenBelt

Selected Year	Storm Water Runoff Catchment Area I	Storm Water Runoff Catchment Area II	Storm Water Runoff Catchment Area III	Storm Water Runoff Catchment Area IV	Storm Water Runoff Catchment Area V	Storm Water Runoff Catchment Area VI	Storm Water Runoff Catchment Area VII	Total Storm Water Runoff to GreenBelt Storage Tanks
	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	**[m³]**
Average Year	548.425	1.004.367	1.118.214	2.065.574	425.433	577.964	771.630	**6.511.607**
Dry Year 1952	244.032	424.797	490.215	912.734	187.208	249.734	316.308	**2.825.029**
Wet Year 1943	765.324	1.184.862	1.488.458	2.831.369	573.456	734.718	815.796	**8.375.983**

The stormwater runoff from the city and the GreenBelt has to be collected and stored in large tanks close to the boundary of the GreenBelt in each of the seven catchment areas.

The required storage volume depends on the infiltration capacity of the facility for groundwater recharge at the proximity of the Matrimandir Lake. The infiltration capacity has been estimated to be 4 M m³/a, 6 M m³/a, 8 M m³/a (see table 5.12).

The required storage volume of the tanks depends also on the infiltration- and evapotranspiration losses. The calculation is based on a sealing of the lake bottom of 0,3 m with a permeability of $k_f = 1 \times 10^{-10}$ m/s and on the potential evapo-transpiration from the tank surface.

Table 5.12: Required volume of Stormwater Storage Tanks in Catchment Areas of the GreenBelt with a Groundwater Recharge Capacity of 4 Million m³/a

Selected year	Required storage volume in catchment area I	Required storage volume in catchment area II	Required storage volume in catchment area III	Required storage volume in catchment area IV	Required storage volume in catchment area V	Required storage volume in catchment area VI	Required storage volume in catchment area VII	Required storage volume in GreenBelt
	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]
average year	387,768	784,606	879,432	1,622,799	263,701	438,308	687,453	**5,064,066**
dry year 1952	91,498	180,701	223,594	495,715	48,554	103,477	179,729	**1,323,268**
wet year 1943	457,563	893,820	1,027,822	2,047,245	313,895	506,230	766,981	**6,013,557**

Table 5.13: Required volume of Stormwater Storage Tanks in Catchment Areas of the GreenBelt with a Groundwater Recharge Capacity of 6 Million m³/a

Selected year	Required storage volume in catchment area I	Required storage volume in catchment area II	Required storage volume in catchment area III	Required storage volume in catchment area IV	Required storage volume in catchment area V	Required storage volume in catchment area VI	Required storage volume in catchment area VII	Required storage volume in GreenBelt
	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]
average year	283,554	603,439	673,870	1,416,789	182,265	329,346	575,886	**4,065,149**
dry year 1952	49,554	107,309	124,134	345,715	24,884	59,868	117,905	**829,369**
wet year 1943	349,179	706,563	796,741	1,649,992	224,689	393,483	646,473	**4,767,120**

Table 5.14: Required Stormwater Storage Tanks in Catchment Areas of the GreenBelt with a Groundwater Recharge Capacity of 8 Million m³/a

Selected year	Required storage volume in catchment area I	Required storage volume in catchment area II	Required storage volume in catchment area III	Required storage volume in catchment area IV	Required storage volume in catchment area V	Required storage volume in catchment area VI	Required storage volume in catchment area VII	Required storage volume in GreenBelt
	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]	[m³]
average year	197,310	506,680	554,037	1,244,789	143,010	272,138	495,260	**3,413,222**
dry year 1952	24,813	74,700	90,709	225,265	6,824	41,237	92,721	**556,269**
wet year 1943	239,044	596,630	679,176	1,482,684	165,592	330,025	567,496	**4,060,648**

All natural tanks have to be excavated, deepened and sealed in order to minimize evaporation and infiltration losses. With the use of compacted red soil mixed with Bentonite a sealing of up to $k_f = 1 \times 10^{-10}$ m/s could be achieved (LGA, 26.09.02).

The largest catchment area of 320 ha drains to the Irumbai Tank. The average annual runoff is estimated to be 2,065 M m³/a. The surface area of the Irumbai Tank is 28 ha and its present storage volume is 561,000 m³ with an average depth of 2 m. This tank will have to be enlarged by excavating it so that it has a an average depth of 4,0 m. The impermeability of the sealing will be increased up to a permeability of $k_f = 1 \times 10^{-10}$ m/s. The Irumbai Tank can provide a storage capacity of 1,12 M m³.

Table 5.15: Required volume of Stormwater Storage Tanks in Catchment Areas of the GreenBelt with a Groundwater Recharge Capacity of 8 Million m³/a

Catchment Area	Surface Area of the Storage Tank	Required Depth of the Storage Tank	Required Storage Volume of the Storage Tank	Remarks
	[m²]	[m]	[m³]	
GB-I	75,000	3	225,000	New
GB-II	168,000	3	504,000	existing 148,000 m² / new
GB-III	183,000	3	549,000	existing 143,000 m² / new
GB-IV	280,000	4	1,120,000	existing 561,000 m³ Irumbai Tank / deepened from t = 2 m to 4 m
GB-V	47,000	3	141,000	New
GB-VI	90,000	3	270,000	Alankuppam Tank deepened to t = 3 m
GB-VII	164,000	3	492,000	144,000 m² / new
Total	1,007,000		3,301,000	

As shown in table 5.15 at the maximum groundwater recharge capacity of 8 M m³/a., in the catchment area GB I the entire storage tank has to be constructed and should have a volume of 225,000 m³.

In the catchment area GB II the existing natural storage tanks have to be enlarged by 14,8 ha in order to have a total storage volume of 504,000 m³.

In the catchment area GB III the existing natural storage tanks have to be enlarged by 14,3 ha in order to have a total storage volume of 549,000 m³.

The Irumbai Tank, in catchment area GB IV, could be deepened, from depth of d = 2 m to a depth of d = 4 m in order to have a storage volume of 1,12 Mio m³.

In the catchment area GB V a new storage tank has to be constructed with a storage volume of 141,000 m³.

In the catchment area GB VI if the Alankuppam Tank could be deepened to an average depth of 3 m the required storage volume of 270,000 m³ could be provided.

In catchment area GB VII the existing storage tank has to be enlarged by 14,4 ha in order to have a storage volume of 492,000 m³.

The detailed calculation of the required storage volume of the tanks in the GreenBelt can be obtained from the tables in Annex 2.3.1 –12, 2.1.1-12, 2.5.1-12 for each of the seven catchment areas (see drawing 42.02/1.2.3).

5.5.3 Purification of the Stormwater Runoff

5.5.3.1 Purification Process

The stormwater runoff from a city contains solid waste, silt, organic and inorganic pollutants, heavy metals, pathogens and also has a high content of nutrients (see table 5.4). Before it can be used for groundwater recharge it should be purified to almost drinking water quality.
For the purification process prior to the recharge eight steps are being proposed:

1. Treatment prior to the inflow into the stormwater storage tanks,
2. Treatment in the stormwater storage tanks,
3. Treatment of the outflow from the storage tanks,
4. Treatment in the streams and water bodies of the re-circulation system,
5. Treatment prior to re-circulation,
6. Treatment prior to inflow into the Matrimandir Lake,
7. Treatment in the Matrimandir Lake,
8. Treatment of the outflow from the Matrimandir Lake.

The inflow in the stormwater storage tanks in the GreenBelt should pass through a sedimentation basin (1st treatment) allowing the skimming and removal of floating and volatile substances (like oils and greases), floating materials (like solid waste, leaves and parts of plants) as well as the sedimentation of grit and sand. Within the storage tank, silt and settlable organic pollutants will be removed (2nd treatment) and part of the dissolved organic pollution will be removed by biological treatment (self purification process of the water body).

The outflow from the storage tanks has to be drawn below but near the surface by a floatable intake and then filtered in a rapid sand filter (3rd treatment). After the rapid sand filtration the water has to be pumped to the top of the hill, near the Peace Area to the beginning of the water courses in the parks. Springs could mark the beginning of the water courses, forming a series of rivulets, ponds and waterfalls throughout the city.

These water bodies will cover an area of 20 to 40 hectares and the length of the water courses will be about 20 kilometers. The biocoenosis in these water courses will participate in the purification of the water as the 4^{th} treatment. The water will flow down the hill and can then be re-circulated through the system according to the demand of flowing water and the dimensions of the water bodies chosen by the landscaper.

The re-circulated water has to be collected in basins where floatable and settlable solids can be removed. The return flow could be passed through an additional rapid sand filter if required (5^{th} treatment). After the final re-circulation the water can then be pumped to the Matrimandir Lake. Prior to the final discharge into the lake the water has to be further treated.

In the 6^{th} treatment process most of the remaining organic and inorganic pollution and nutrients will be removed. The water has to be drawn from the collection basins below but near the surface through a floating intake and then filtered using a rapid sand filter. After the rapid sand filtration a biological treatment is required. It can either be a rootzone treatment process (which will increase the evaporation losses) or a slow sand filtration. Up to this stage, two thirds of the original load of phosphorous would have been removed in order to maintain a mesotrophic state in the proposed Matrimandir Lake. The load of phosphorous in the runoff has been roughly estimated to be 1,087 kg/a.

The natural self-purification capacity of the Matrimandir Lake is used as the final stage of purification (7^{th} treatment). The total content of phosphorous should remain below P_{tot} = 20 µg/l and should not exceed P_{tot} = 30 µg/l. The remaining nutrients, dissolved, particular and complex pollutants would be absorbed by the biocoenosis. After decomposition and mineralization of the organic matter of the biocoenosis the remains are deposited at the bottom of the lake. The organic load and the growth of biomass within the lakes should be low so that dissolved oxygen does not get depleted and the surface of the lake bottom remains aerobic.

Since the outflow of a lake contains its biocoenosis, organic filterable solids like alga, zooplankton and bacteria that could clog the infiltration trenches, it is recommended that a rapid sand filtration be used to remove most of the solids (8^{th} treatment).

Figure 5.6 Purification of Stormwater Runoff

5.5.3.2 Dimensioning of the sedimentation basin (1st Treatment)

The inflow of stormwater into storage tanks that are located in the GreenBelt has to pass through sedimentation basins where floatable substances are skimmed and settlable solids like grit and sand are removed.

The sedimentation basin can be constructed as separate structures or integrated into the inflow structures of the storage tanks. The sedimentation basins should provide a retention time of 15 minutes, or for large inflows maintain a velocity of 0,3 m/s to allow the sedimentation of grit.

The required surface area and volume of the sedimentation basin can be obtained from table 5.16.

Table 5.16: Sedimentation Basin at the Stormwater Storage Tanks

		Alternative I		Alternative II					
Catchment Area	Average Inflow	**Sedimentation Time**	Required Volume of the Sedimentation Basin	**Alternative II 0,3 m/s Velocity of Flow**	Required Area of Discharge Cross Section of the Sedimentation Basin	Required Depth of the Cross Section	Required Width of the Cross Section	Required Length of the Basin	Required Surface of Sedimentation Basin
	Q	t_{sed}	V_I	v	A	H	B	L	A_Q
	[l/s]	min	[m³]	[m/s]	[m²]	[m]	[m]	[m]	[m²]
GB II	9,129	15,00	8,216	0,30	30	1	30	25	750
GB III	1,923	15,00	1,731	0,30	6	1	6	25	150
GB IV	26,697	15,00	24,027	0,30	89	1	89	25	2,225

5.5.3.3 Dimensioning of the Stormwater storage tanks in the GreenBelt (2nd Treatment)

The inflow to the stormwater storage tanks contains all the dissolved pollution and most of the settleable solids. Only floatable solids, grit and sand would have been removed during the 1^{st} treatment in the sedimentation basin. The purification efficiency will be determined by the retention time of the polluted water in the tank. The shortest retention time can be estimated in a wet year and during the highest recharge and infiltration rate of 8 M m³/a.

Table 5.17: Retention time in the Stormwater Storage Tanks (Infiltration Rate 8 M m³/a)

	Retention Time in Catch-ment Area I	Retention Time in Catch-ment Area II	Retention Time in Catch-ment Area III	Retention Time in Catch-ment Area IV	Retention Time in Catch-ment Area V	Retention Time in Catch-ment Area VI	Retention Time in Catch-ment Area VII
month	[month]	[month]	[month]	[month]	[month]	[month]	[month]
August	1,0	1,0	1,0	1,0	1,0	1,0	1,0
September	1,0	1,0	1,0	1,0	1,0	1,0	1,0
October	1,9	2,1	2,2	2,8	1,4	2,0	2,9
November	3,1	3,7	3,8	4,8	2,4	3,5	5,3
December	0,9	0,9	1,1	1,4	0,7	1,0	1,2
January	0,1	0,0	0,1	0,1	0,0	0,0	0,0
February	0,0	0,0	0,0	0,0	0,0	0,0	0,0
March	0,0	0,0	0,0	0,0	1,0	0,0	0,0
April	0,9	0,0	0,0	0,0	1,0	0,0	0,0
May	1,0	1,0	0,2	0,1	1,0	1,0	0,0
June	1,0	1,0	1,0	0,1	1,0	1,0	0,0
July	1,0	1,0	1,0	0,4	1,0	1,0	0,3

5.5.3.4 Dimensioning of the Rapid Sand Filter at the Stormwater Storage Tanks (3rd Treatment)

For the removal of filterable solids from the stored stormwater runoff a rapid sand filter is considered as an appropriate and suitable technology.

The filter should have the following properties:

one layer filter
thickness: 2,000 mm
total height of filter: 4,5 m
filter material: quartz sand Ø 0,5 to 2 mm
filter velocity: 15 m/h
operational pressure: 0,6 – 0,8 bar
volume of backwash: 1 - 2 % of filtrate
volume of backwash: a) air and water: 3 minutes 60 $m^3/m^2 \cdot h$ air
10 $m^3/m^2 \cdot h$ water
b) water: 5 minutes water 40 - 90 $m^3/m^2 \cdot h$

The dimensioning of the rapid sand filter unit will be calculated for the maximum stormwater flow in the catchment area GB-IV at 193.000 m³/month.

Q_t = 193.000 m³/month = 268,06 m³/h
Q_F = 268,06 m³/h
v = 10 m/h
A_{SF} = Q / v
A_{SF} = 268,06 m³/h / 10 m/h
A_{SF} = 26,81 m²

A closed and standing rapid sand filter unit with a diameter of 3 m will be used for filtration. The surface A_S of the filter can be estimated at

D = 3 m.
A_S = $\pi/4 \times d^2$
A_S = 7,07 m²

The required number of rapid sand filter units can be calculated from the total required filter surface:

A_{SF} = 26,81 m² with:

Z = 26,81 m² / 7,07 m² = 3,79.

The backwash time after an operation time of 20 to 100 hours should last for 10 to 20 minutes. The time for the repairs and maintenance of the filters have to be included, so it is ensured that at least 4 numbers of rapid sand filters have to be provided for catchment area IV. The requirement for the other catchment areas can be derived from table 5.18.

At the Irumbai Tank the water has to be drawn by a floating intake and pumped by 3 pumps with a capacity of Q = 89 m³/h each (Q = 24,2 l/s), giving altogether 268 m³/h. The pumping head can be estimated to be 10-20 m (pressure at the filter will be 0,6-0,8 bar, difference in elevation and friction losses in the pipes will be the equivalent of 1-2 m).
For the other remaining tanks the pumps required are shown in table 5.19.

Table:5.18: Required Number of Rapid Sand Filters at the Stormwater Storage Tanks

Catchment Area	Average Inflow	Average Outflow	Required Filter Surface with v = 10 m/h	Number of Filter with a Diameter D = 3m	Number of Filter with a Diameter D = 2m	Chosen Filter
	[m³/month]	[m³/h]	[m²]			[m²]
Catchment Area I	77,000	106.94	10.69	1.51	3.40	2 x DN 3000
Catchment Area II	135,000	187.50	18.75	2.65	5.97	3 x DN 3000
Catchment Area III	135,000	187.50	18.75	2.65	5.97	3 x DN 3000
Catchment Area IV	193,000	268.06	26.81	3.79	8.53	4 x DN 3000
Catchment Area V	77,000	106.94	10.69	1.51	3.40	2 x DN 3000
Catchment Area VI	77,000	106.94	10.69	1.51	3.40	2 x DN 3000
Catchment Area VII	77,000	106.94	10.69	1.51	3.40	2 x DN 3000

Table:5.19: Required Pumps to feed the Rapid Sand Filter at the Stormwater Storage Tanks

Catchment Area	Average Inflow	Average Outflow	Chosen Number of Pumps	Including one additional Standby Pump	Estimated Pumping Head	Required Power for one Pump
	[m³/month]	[l/s]	[mm]	[m]	[m]	[kW]
Catchment Area I	77,000	29.71	2	3	15.00	4.46
Catchment Area II	135,000	52.08	2	3	15.00	7.81
Catchment Area III	135,000	52.08	2	3	15.00	7.81
Catchment Area IV	193,000	74.46	3	4	18.00	8.94
Catchment Area V	77,000	29.71	2	3	15.00	4.46
Catchment Area VI	77,000	29.71	2	3	15.00	4.46
Catchment Area VII	77,000	29.71	2	3	15.00	4.46

5.5.3.5 Dimensioning of the Booster Pumps for the Feeding of the Re-circulation system from the Stormwater Storage Tanks in the GreenBelt

The discharge from the stormwater storage tank will be purified by rapid sand filtration and can then be lifted from the GreenBelt into the city to charge the water courses of the re-circulation system.

The booster pumps have to be dimensioned to a maximum load of 8 M m³/a.
The maximum discharge will be from catchment area IV:

$$Q_{Sp} = 193{,}0\ m^3/h$$
$$Q_{Sp} = 74{,}5\ l/s$$

The head loss can be estimated from the difference in elevation and the friction losses during the conveyance of the water.

The pressure pipe lines have to be laid from the pump station at the stormwater storage tanks across the GreenBelt and through the radial roads to the centre of the city near the Matrimandir Lake.
The roughness of the pipe is estimated to be

$$k_b = 0{,}1\ mm$$

The velocity in the pipe should be v> 0,8 m/s to prevent sedimentation,
the velocity is chosen to be

$$v = 1{,}06\ m/s \text{ and}$$

the pipe diameter is chosen to be

DN 250

the discharge is chosen to be

$$Q = 75{,}0\ l/s$$

the friction losses are estimated to be

$$H_v = 2.500\ m \times 2{,}3\ mm/m = 5{,}75\ m.$$

the difference in elevation is estimated to be

$$H_{geod} = H_1 - H_2$$
$$H_{geod} = 50{,}50\ m\ NN - 28{,}00\ m\ NN$$
$$H_{geod} = 22{,}5\ m.$$

the pumping head is estimated to be

$$H_{ges} = H_{geod} + H_v$$
$$H_{ges} = 22{,}5\ m + 5{,}75\ m$$
$$H_{ges} = 28{,}25\ m.$$

The pumping station will be designed for a pumping head of H = 29,00 m with two parallel operating pumps each of a capacity of

$$q = 37{,}5 \text{ l/s}.$$

The energy requirement of the pump can be estimated to be

$$g = 9{,}81 \text{ m/s}^2$$

$$\rho = 1000 \text{ kg/m}^3$$

$$H = 29{,}0 \text{ m}$$

$$Q_{Pmax} = 37{,}5 \text{ l/s}$$

$$\eta_P = 0{,}6$$

$$P_P = \frac{Q_{max} \cdot H \cdot g \cdot \rho}{1000 \cdot \eta_P}$$

$$P_P = \frac{37{,}5{,}0\text{l/s} \cdot 29\text{m} \cdot 9{,}81\text{m/s}^2 \cdot 1\text{kg/l}}{1000 \cdot 0{,}6}$$

$$P_P = 18{,}2 \text{ kW}$$

energy requirement of the engine P_M can be estimated with an efficiency coefficient of 0,86 of the energy requirement of the pump

$$P_M = \frac{P_P}{0{,}86} = 21{,}1 \text{ kW}.$$

The energy requirement for the booster pumps at the stormwater storage tanks in the GreenBelt to lift the water to the city centre and into the re-circulation system can be obtained from table 5.20.

Table 5.20 Dimensioning of the pipes and booster pumps to lift the stormwater from the Stormwater Storage Basin to the City Centre and into the Re-circulation System

Catchment Area	Average Inflow	Average Outflow	Chosen Pipe Diameter	Pipe Length to Matrimandir Lake	Chosen Pipe Flow	Velocity	Friction Losses in Pipe	Total Head Losses	Total Difference in Elevation	Number of Pumps chosen	Power Requirement for one Pump
	[m³/month]	[l/s]	[mm]	[m]	[l/s]	[l/s]	[mm/m]	[m]	[m]	[piece]	[kW]
Catchment Area I	77,000	29.71	200	250.000	30	0.95	4.50	11.25	33.75	1	19.62
Catchment Area II	135,000	52.08	250	250.000	53	1.08	3.80	9.50	32.00	2	16.43
Catchment Area III	135,000	52.08	250	250.000	53	1.08	3.80	9.50	32.00	2	16.43
Catchment Area IV	193,000	74.46	300	250.000	75	1.06	2.30	5.75	28.25	2	20.53
Catchment Area V	77,000	29.71	200	250.000	30	0.95	4.50	11.25	33.75	1	19.62
Catchment Area VI	77,000	29.71	200	250.000	30	0.95	4.50	11.25	33.75	1	19.62
Catchment Area VII	77,000	29.71	200	250.000	30	0.95	4.50	11.25	33.75	1	19.62

5.5.3.6 Purification of the Stormwater runoff in the water courses and water bodies in the parks

All parks and public green spaces will be provided with an interconnected system of watercourses, waterfalls, fountains and ponds and other water bodies. The outflow from the stormwater storage pond after the rapid sand filtration will be re-circulated through this system. There will be a constant flow of water through this system, which will aerate the water as it flows through. Simultaneously the biocenosis (biofilm and suspended biomass) of all the water bodies in this system will further purify the re-circulated water, and this can be considered as a 4th treatment that uses biological processes. There are 4 parks in the city, and each will be provided with an interconnected system of water courses and ponds.

The length of the water courses can be estimated to be at least

$$L = 20{,}000\ m^{*}.$$

The surface of the ponds and other water bodies can be estimated to be up to

$$A = 40\ ha^{*}.$$

The retention time can be estimated to be

$$t = 400{,}000\ m^2 \times 1{,}5\ m + 20{,}000\ m \times 2{,}0\ m \times 0{,}5\ m / 0{,}48\ m^3/s$$

$$t = 14{,}9\ d$$

if the water passes the system at least twice.

With the use of this system the nutrients, especially phosphorus, can be transformed into biomass. If the retention time is more than 10 days (BERNHARDT, 1979) up to 70 % of the phosphorus could be removed. This can be achieved, if the collected stormwater runoff is pumped from the seven storage tanks into a central distribution pipe encircling the Matrimandir Lake and distributing the flow uniformly into the re-circulation system into the four parks. If the water requires more than 10 days for the flow through the interconnected water courses and ponds, the phosphorus will be incorporated into the biomass of the aquatic eco-system. It can remain in the system or it will be filtered prior to the discharge from this system into the Matrimandir Lake.

The water bodies will have to be sealed to minimize infiltration losses. The large surface of the water courses will also lead to substantial evapo-transpiration losses, through this will help to reduce the air temperatures in the city. The losses may be in the same range as those from evapo-transpiration, if this area in the parks is covered by dense vegetation. (see drawing 42.02/1.2.6).

* The water courses and water bodies throughout the city have not been designed yet. In the model of the presentation the water bodies cover an area of more than 100 ha.

5.5.3.7 Dimensioning of the Collecting Basins, the Booster Pumps and the Rapid Sand Filters of the Re-circulation System

It can be assumed that the water that is discharged from the stormwater storage tanks will be re-circulated at least once. There will be 4 independent re-circulation units, one in each park. If the distribution of the flow is uniform, the flow will be about 120 l/s in each of the streams. The pipeline for the re-circulated water has to have a diameter of at least DN 400 mm. The required pumps and sumps can be obtained from table 5.21.

If it is found that prior to re-circulation a further purification is required, a rapid sand filtration could be integrated in the re-circulation process. For the re-circulation system the filter capacity has to be the same for the stormwater storage tanks as shown in tables 5.18 and 5.21.

Table 5.21: Dimensioning of the Re-circulation Pump

Re-circulation Zone	Average Flow	Chosen Pipe Diameter	Pipe Length	Pipe Velocity	Pipe Losses	Total Pipe Losses	Difference in Altitude	Total Head Losses	Number of Chosen Pumps	Power Requirement of one Pump	Volume of Pump Sump
	[l/s]	[mm]	[m]	[l/s]	[mm/m]	[m]	[m]	[m]		[kW]	m³
North	120	400	1400	0,95	2,2	3,08	8,00	11,08	1,00	26	8,00
East	120	400	1200	0,95	2,2	2,64	10,00	12,64	1,00	29	8,00
North-West	120	400	1400	0,95	2,2	3,08	22,00	25,08	1,00	58	8,00
South-West	120	400	1300	0,95	2,2	2,86	11,00	13,86	1,00	32	8,00

Table 5.22: Required number of Rapid Sand Filters for the Re-circulation

Re-circulation Zone	Average Discharge	Average Discharge	Required Filter Surface with v = 10 m/h	Number of Filter with a Diameter D = 3 m	Number of Filter with a Diameter D = 2 m	Chosen Filter
	[l/s]	[m³/h]	[m²]			[m²]
North	120,00	432,00	43,20	6,11	13,75	7 x DN 3000
East	120,00	432,00	43,20	6,11	13,75	7 x DN 3000
Northwest	120,00	432,00	43,20	6,11	13,75	7 x DN 3000
Southwest	120,00	432,00	43,20	6,11	13,75	7 x DN 3000

This additional filtration may not be required but should be considered an option if additional pollution will enter the streams in the city. The re-circulation increases the flow through the parks from 75 l/s to 120 l/s.
The amount of water discharged into the Matrimandir Lake for groundwater recharge will be the same amount that is discharged from the stormwater storage tanks. The water discharged to the Matrimandir Lake has to undergo further biological treatment, through rootzone treatment plants or through slow sand filtration. To minimize the maintenance of the slow sand filter, a rapid sand filtration prior to the slow sand filtration can be proposed (as shown in table 5.23).

Table 5.23: Required number of Rapid Sand Filters after the Re-circulation prior to the slow sand filtration

Re-circulation Zone	Average Discharge	Average Discharge	Required Filter Surface with v = 10 m/h	Number of Filter with a Diameter D = 3 m	Number of Filter with a Diameter D = 2 m	Chosen Filter
	[l/s]	[m³/h]	[m²]			[m²]
North	75,00	270,00	27,00	3,82	8,59	4 x DN 3000
East	75,00	270,00	27,00	3,82	8,59	4 x DN 3000
Northwest	75,00	270,00	27,00	3,82	8,59	4 x DN 3000
Southwest	75,00	270,00	27,00	3,82	8,59	4 x DN 3000

5.5.3.8 Dimension of the Booster Pumps of the Rapid and Slow Sand Filter at the Re-circulation system

The re-circulation requires collection sumps and booster pumps to lift the water at the outer ring road through the rapid sand filters into the slow sand filters. The required size of the collection sump and the booster pumps can be obtained from table 5.23.

Table 5.24 Required pumps to feed the Rapid and Slow Sand Filter at the Collection Sump of the Re-circulation system

Re-circulation zone	Average Discharge	Chosen Number of Pumps	Including one additional Standby Pump	Required Collection Sumps	Estimated Pumping Head	Required Power for one Pump
	[l/s]	[-]	[-]	[l/s]	[m]	[kW]
North	75	3	4	4,50	15.00	7.50
East	75	3	4	4,50	15.00	7.50
Northwest	75	3	4	4,50	15.00	7.50
Southwest	75	3	4	4,50	15.00	7.50

From the rapid sand filter the slow sand filter will be charged by gravity.

It can be assumed that the water that is discharged from the stormwater storage tanks will be re-circulated at least once. If it is found that prior to the re-circulation a further purification is required, a rapid sand filtration could be integrated in the re-circulation process. For the re-circulation the filter capacity has to be the same as that of the stormwater storage tanks as shown in tables 18 and 20.

5.5.3.9 Dimensioning of the Slow Sand Filter in the Re-circulation System

For the removal of organic and inorganic substances biological treatment is required and a slow sand filter is proposed for this purpose.
The filter should have a depth of 1,000 mm, the size of the grains should be 0,8 - 1,2 mm, and the velocity of the filtration should be v = 0,1 m/h. The maximum depth of the water above the filter should be 0,6 - 1,0 m. The dimensioning of the slow sand filter is based on the annual recharge capacity of 4 Mio m³/a. During a year with high precipitation giving a recharging capacity of 8 Mio m³/a only half of the flow can pass through the slow sand filter. Or alternatively the filtration capacity has to be raised to 0,2 m/h.
It can be expected that during periods of heavy rainfall the dilution will reduce the concentration of the pollution substantially and therefore the flow through the filters can be increased.
The slow sand filter for the re-circulation system of the north zone can be estimated as follows:

$$
\begin{aligned}
Q_{tIV} &= 40 \text{ l/s} \\
Q_{tIV} &= 144{,}0 \text{ m}^3\text{/h} \\
v &= 0{,}1 \text{ m/h} \\
A_{SF} &= Q / v \\
A_{SF} &= 144{,}0 \text{ m}^3\text{/h} / 0{,}1 \text{ m/h} \\
A_{SF} &= 1.440 \text{ m}^2
\end{aligned}
$$

A covered sand filter with a filter surface area of 37,5 m x 40 m is recommended.
The dimensions of the slow sand filter for the remaining catchment areas can be obtained from Table 5.25.

Table 5.25 Dimensioning of the Slow Sand Filters at the Re-circulation system (4 Mio m³/a)

Re-circulation Zone	Average Discharge	Average Discharge	Slow Filter Area with v = 0.1 m/h	Chosen Filter Area
	[l/s]	[m³/h]	[m²]	[m²]
North	40,00	144,00	1440,00	1500
East	40,00	144,00	1440,00	1500
Northwest	40,00	144,00	1440,00	1500
Southwest	40,00	144,00	1440,00	1500

5.5.3.10 Dimensioning of the Booster Pumps for the feeding of the Matrimandir Lake from the Re-circulation System

The discharge from the stormwater storage tanks will be purified by rapid sand filtration and will then be lifted from the GreenBelt into the city centre to charge the water courses of the re-circulation system. At the outer ring road the re-circulated water has to be collected in a sump and then purified through a rapid sand filtration and a slow sand filtration. After the final purification the water can be pumped into the Matrimandir Lake.

The booster pumps have to be dimensioned for a maximum load of 8 M m³/a.
The discharge will be from the re-circulation system of the “North” zone:

$$Q_{sp} = 75,0 \text{ l/s}$$

The head loss can be estimated from the difference in elevation and the friction losses during the conveyance of the water. The pressure pipelines have to be laid from the pumping station at the collection sumps of the re-circulation system of the Radial Road to the centre of the city near Matrimandir Lake.

The roughness of the pipes is estimated to be

$$k_b = 0,1 \text{ mm}$$

The velocity in the pipe should be v > 0,8 m/s to prevent sedimentation,
the velocity is chosen to be

$$v = 0,78 \text{ m/s}$$

and the pipe diameter is chosen to be DN 350.
the discharge is chosen to be

$$Q = 75,0 \text{ l/s}$$

the friction losses are estimated to be

$$H_v = 1{,}400 \text{ m x } 1{,}8 \text{ mm/m} = 2{,}52 \text{ m.}$$

The difference in elevation is estimated to be

$$H_{geod} = H_1 – H_2$$
$$H_{geod} = 50{,}50 \text{ m NN} – 41{,}50 \text{ m NN}$$
$$H_{geod} = 9{,}00 \text{ m.}$$

The pumping head is estimated to be

$$H_{ges} = H_{geod} + H_v$$

The pumping station will be designed for a pumping head of H = 12,00.m with two parallel operating pumps each with a capacity of

$$q = 37{,}5 \text{ l/s.}$$

The energy requirement of the pump can be estimated to be

$$g = 9{,}81 \text{ m/s}$$
$$p = 1{,}000 \text{ kg/m}^3$$
$$H = 11{,}52 \text{ m}$$
$$Q_{pmax} = 75{,}00 \text{ l/s}$$
$$\eta_p = 0{,}6$$
$$P_P = \frac{Q_{max} \cdot H \cdot g \cdot \rho}{1000 \cdot \eta_P}$$
$$P_p = \frac{75 \text{ l/s} \cdot 11{,}52 \text{ m/s} \cdot 9{,}81 \text{ m/s}^2 \cdot 1 \text{ kg/l}}{1000 \cdot 0{,}6}$$
$$P_p = 14{,}12 \text{ kW}$$

The energy requirement of the engine P_M can be estimated to have an efficiency coefficient of 0,86 of the energy requirement of the pump

$$P_M = \frac{P_P}{0{,}86} = 16{,}43 \text{ kW.}$$

The energy requirement for the booster pumps to lift the stormwater from the Collection Sumps of the Re-circulation System into the Matrimandir Lake can be obtained from table 5.26.

Table 5.26 Dimensioning of the pipes and booster pumps to lift the stormwater from the Collection Sumps of the Re-circulation System into the Matrimandir Lake

Re-circulation Zone	Average Outflow	Chosen Pipe Diameter	Pipe Length to Matrimandir Lake	Chosen Pipe Flow	Velocity	Friction Losses in Pipe	Total Head Losses	Total Difference in Elevation	Number of Pumps chosen	Power Requirement for one Pump Engine
	[l/s]	[mm]	[m]	[l/s]	[l/s]	[mm/m]	[m]	[m]	[piece]	[kW]
North	75.00	350	1400	75	0,78	1.80	2,52	11,52	1	17
East	75.00	350	1200	75	0,78	1.80	2,16	13,16	1	19
North-West	75.00	350	1400	75	0,78	1.80	2,52	25,52	1	37
South-West	75.00	350	1300	75	0,78	1.80	2,34	14,34	1	21

5.5.3.11 Purification of the Stormwater Runoff in the Matrimandir Lake (7th Treatment)

At this stage when the stormwater runoff is pumped from the re-circulation system into the Matrimandir Lake it has undergone 6 stages of treatment. By now the filterable solids have been removed from the water, and through biological treatment part of the organic and inorganic pollution in the water has been removed. The remaining organic and inorganic pollution will then be assimilated by the biocenosis of the Lake or transformed in less harmful components that would be removed for example by fish or other organisms or deposited on the bottom of the Lake. The Lake therefore becomes a Freshwater Eco-System. The biological and biochemical properties of the lake water will be influenced by the geological formations of the lake basin and its catchment area, by its morphology, the vegetation and anthropogenic influences at the lake or in the catchment area.

The design of the Matrimandir Lake has taken the following consideration into account:
The morphology of the lake, its depth, the level of water fluctuation, the depth of the outlet, the location of the inlet all influence the retention time. The exchange of water, the stratification and mixing, the temperature of the water, its appearance and the effect of density currents as well as the fish productivity all influence the biological and biochemical properties of the lake. The location and orientation of the lake in

respect to the main wind direction and the exposure of the surface of the water to the wind influences the mixing, the stratification, the aeration and oxygen content as well as the distribution of macrophytes and alga. The amount, size and growth of vegetation in side arms or shallow water zones influences the development and the distribution of alga, macrophytes and the agents of water-borne diseases.

The annual fluctuation of the temperature and density of the inflow into the lake influences the retention time, the development of density currents, the mixing and the development of gases from anaerobic processes. In an aquatic eco-system the primary production of autotrophic phytoplankton and higher plants (macrophytes) will be consumed by zooplankton and fish that feed on it or on the detritus (particular organic matter) they generate. The degradation of the primary production (autotrophic process) of particulate or dissolved organic substrate is taken care of by heterotrophic organisms like bacteria and fungi.

Both autotrophic and heterotrophic processes are metabolically coupled through a food web. The intensity of both processes depends on the availability of nutrients, and the effects are much stronger in stagnant water than flowing water. The influences on the water quality are much more complex in the tropics and subtropics than in temperate zones. Besides the nutrients, the light can also be a limiting factor. Temperature, density and viscosity of the lake water influence the sedimentation of certain types of alga and with it the availability of nutrients as well as the food chains from the zooplankton to the fish.

Most of the available information on lakes is based on lakes in temperate climates. The function of lakes in the tropics and subtropics shows fundamental differences to lakes in temperate zones. The characteristic differences are the extremes in the availability of water. In the tropics and subtropics the large volume of inflow during the rainy season usually contains large amounts of erosion material, of organic and inorganic nature, causing rapid changes in the water quality. The nature of the eroded soils are often kaolonitic, iron rich clay suspensions in colloidal distribution. The particles can hardly be settled, but change the penetration of light and the photosynthetic conditions decisively. Tropical soils are poor in nutrients, so the primary production in the lake can therefore be limited by nitrogen, unlike in temperate zones, where phosphorous generally is the limiting nutrient.

In the tropics and subtropics humic acids are washed out very rapidly from the soils by rains that have high temperatures and result in the lowering of ion concentrations significantly once they reach the lake water. The humic acids become a very vital factor for all biochemical processes.

In the tropical and subtropical regions the temperature plays a decisive role. The degradation of organic matter can be higher by several degrees while at the same time the dissolved oxygen is decreasing with higher temperature. Depletion of oxygen in the lake water can develop rapidly. Even small quantities of nutrients can lead to high levels of primary production. Tropical lakes contain much less dissolved oxygen than temperate lakes due to higher water temperatures. Oxygen is required for the degradation of organic substances . Since the temperature of the inflow varies in the course of the year substantially, destined density currents can develop due to the high water temperatures in the lake.

At high water temperatures even small differences in temperature lead to relatively large differences in the density of the water, and can therefore lead to stratification and stagnation over a long period where already small quantities of organic material can deplete the oxygen.

For the supply of oxygen to the lower layers of water in the lake, the occurrence of circulation and stagnation is the key to the understanding of the function of tropical or subtropical lakes. The specific properties of density, viscosity and high temperature in tropical and subtropical lakes result in stratification cycles which can hardly be compared with a lake in the temperate zone.

The phytoplankton are produced in the tropholythic zone, the epilymnion where light can penetrate. Below this zone, degradation of organic matter takes place; also at the lake bottom where the degradation and mineralization of the sediments is taken over by the benthos in the bentic zone.

The littoral benthos as well as the sub-littoral benthos is composed of a high diversity of groups and species with a considerable annual production. This is a zone of high microbial activity which is connected with the entire metabolism of the lake. The bottom fauna is very intolerant to low oxygen levels.

Lakes are classified in four stages of trophy, based on their productivity from low to high, as olygotroph – mezotroph - eutroph and hyper-eutroph. In designing the lake the main concern is to prevent its eutrophication and to keep the lake in the ideal state of oligotrophy or mezotrophy.

The biochemical cycles in the lake are controlled by morphometric parameters, such as the "mean depth", by hydrographical parameters like the "residence time of the water", the loading from the catchment such as the "retention" on the one side and on the other side by the "external and internal loading" of nutrients.

According to VOLLENWEIDER (1968) the expected trophic conditions in water bodies depend on the quantitative loading of phosphorus (P) and nitrogen (N) as the initial growth limiting factors. The critical N : P ratio is considered to be 10 : 1 (in phytoplankton the mass ratio is 7.2 : 1). If the ratio is > 7 only then nitrogen is considered the growth limiting factor; in the range of 7 to > 12 phosphorus is the limiting factor, which is generally the case, and what has to be expected also for the lakes in Auroville due to the anthropogenic influence in the catchment area.

The centre of the city is located at $12°00^{31}$ N and $79°48^{51}$ E at 52 m SL. If the depth of the lake is less than 20 m the lake will be classified according to the stratification as continuously warm polymictic where many phases of circulation have to be expected (LEWIS, 1983). Water temperature and density of the water may determine the mixing pattern to be more irregular and less continuous. The water temperature at the lake bottom can be estimated according to the equation of LEWIS (1987):

$$T_b = 28{,}9 - 0{,}43\ B' - 0{,}0038\ H$$
$$T_b = 28{,}9 - 0{,}43\ (12°N - 3{,}4\ °N) - 0{,}0038 \times 52\ m$$
$$T_b = 28{,}9 - 3{,}698 - 0{,}1976$$
$$T_b = 25{,}0°C$$

B' = corrected latitude
H = elevation above mean sea level

The oxygen saturation will be at 8,15 mg O_2/l, equal to 74,8 % of saturation of lakes in temperate climates at 10 °C

The intensity of the primary production will depend on the growth limiting factors, the nutrients, phosphorus (P) and nitrogen (N) or the light. Since the inflow of the lake would have undergone a substantial pre-treatment there will be no light-limiting substances in the water.

Tropical lakes are known for their high bio-availability of nutrients because of the potential for their re-circulation. The Matrimandir Lake can only fulfill its designed function if its eutrophication is prevented and its trophy-status remains mesotrophic, which can be achieved if the total phosphorus content remains below

$$30\ \mu g\ P_{tot}/l$$

in the lake water.

Only then can it be expected that oxygen will not be depleted down to the lake bottom, and the appearance of anoxic or anaerobic zones in the water will be prevented.

The potential phosphorus load of the stormwater runoff in Auroville has been estimated according to the available data in the literature, and the mean annual load can be obtained in table 5.27 at a rate of approximately 1,087 kg P_{tot}/a.

From the literature the average annual phosphorus load of the stormwater runoff has been indicated in table 5.27 to be

1,086.73 kg/a

The phosphorus load can also be estimated on general assumption as:

1. Phosphorus load from precipitation on the lake surface
   18,6 ha · 0,2 kg/ha·a = 3,72 kg P/a
2. Phosphorus load from commercial areas
   126 ha · 0,8 kg/ha·a = 100,8 kg P/a
3. Phosphorus load from roads and yards
   154 ha · 0,5 kg/ha·a = 77,0 kg P/a
4. Phosphorus load from agricultural land
   603 ha · 0,4 kg/ha·a = 241 kg P/a
5. Phosphorus load from forest areas
   603 ha · 0,05 kg/ha·a = 30,15 kg P/a
6. Phosphorus load from public green areas
   104 ha · 0,05 kg/ha·a = 5,20 kg P/a

The total phosphorus load according to this estimate will be

458,07 kg P/a

see table 5.29.

The critical phosphorus load can be estimated according to the equations presented in the study of the OECD (1982). During the past planning process five different options of the lake had been designed and morphologic data were taken from there.

Table 5.27: Pollution of Stormwater Runoff in different Towns in Germany and Estimate of the Pollution in Auroville

		Source of Information							Author of Information							Estimated Pollution in Auroville			
Pollutant	Unit	Pullach	Hildesheim P4	Hildesheim P5	A 81	A6	A 8 / B 10	Rainfall in open landscape	Essen-K.	Hellmann	Peukert	Ruppert	Göttle	Klein	Merkel	unit	AV min	**AV average**	AV max
filterable solid	kg/ha*a	430		2121	873	848	479						12.5	12.1		kg/a	19,464.06	**1,097,432.88**	3,411,840.60
settlable solid	l/ha*a	2240		11443												l/a	3,603,264.00	**11,005,236.90**	18,407,209.80
KMnO4	kg/ha*a		118.8	596												kg/a	191,101.68	**574,913.64**	958.725,60
BOD5	kg/ha*a	33	26.9	93.3						4						kg/a	6,434.40	**63,217.98**	150,082.38
COD	kg/ha*a	331	170.3	568.5	672	557	207		13.8	5	25.6	20				kg/a	8,043.00	**413,442.37**	1,080,979.20
TOC	kg/ha*a			176.9												kg/a	284,561.34	**284,561.34**	284,561.34
Chloride as Cl	kg/ha*a	86	77.2	501.2	1011	777	1344			7		0.87		1.2	1.2	kg/a	1,399.48	**612,340.94**	2,161,958.40
Sulphate (as SO4-)	kg/ha*a			239.9						17	41	8.8			7.3	kg/a	11,742.78	**101,020.08**	385,903.14
PO4-ges	kg/ha*a	4.4	4		1.62	1.45	0.63				0.33					kg/a	530.84	**3,332.48**	7,077.84
Total Phosphorus as P	kg/ha*a															kg/a	173.11	**1,086.73**	2,308.10
NH4 - N	kg/ha*a			14.4	4.6	3.22	1.03	1.3	2.9				1.58	1.2		kg/a	1,656.86	**6,078.50**	23,163.84
NO2 - N	kg/ha*a	0.4		0.7									0.024			kg/a	38.61	**602.69**	1,126.02
NO3 - N	kg/ha*a	10.6		11.4									3.57		5.6	kg/a	5,742.70	**12,535.02**	18,338.04
Iron as Fe	kg/ha*a			64.7	23.37	28.81	4.37			0.6		0.043		0.22		kg/a	69.17	**28,061.57**	104,076.42
Lead as Pb	kg/ha*a			1.9	1.332	1.155	0.36	22	35		0.059	0.005		0.029	0.0121	kg/a	8.04	**9,949.53**	56,301.00
Cadmium as Cd	kg/ha*a			0.0264	0.037	0.029	0.0072									kg/a	11.58	**40.05**	59.52
Chromium as Cr	kg/ha*a			0.1514	0.062	0.1	0.012									kg/a	19.30	**130.86**	243.54
Copper as Cu	kg/ha*a			0.8	0.621	0.544	0.13	2	29		0.002	0.007		0.012	0.0355	kg/a	3.22	**5,332.75**	46,649.40
Zinc as Zn	kg/ha*a			2.7	2.329	2.892	0.715	0.031	0.114	0.145	0.048			0.1	0.0945	kg/a	49.87	**1,474.84**	4,652.07
Nickel as Ni	kg/ha*a			0.2					2.7							kg/a	321.72	**2,332.47**	4,343.22
Mercury as Hg	kg/ha*a			0.0025												kg/a	4.02	**4.02**	4.02
PAK	kg/ha*a			0.0015	0,018	0,014	0.005									kg/a	2.41	**15.48**	28.95
Mineral oil	kg/ha*a			5.2	43,27	27,09	4.85									kg/a	7,801.71	**32,336.88**	69,604.12

Table 5.28: Optional Morphological Parameters for the Matrimandir Lake

Alternative	Mean Depth t_M	Maximum Depth t_{Max}	Surface Area A_o	Volume V
	m	M	m²	m³
1	5,71	10	89.322	510.250
2	6,90	10	137.027	945.610
3	7,60	10	181.102	1.376.280
4	7,67	10	186.000	1.426.000
5	8,93	15	186.000	1.660.000

The critical specific phosphorus load can be estimated according to the following equation:

$$L_c = \left[P_c^{sp}\right] * q_s \left(1 + \sqrt{\frac{\bar{z}}{q_s}}\right)$$

L_c $= critical\ specific\ Phosphorus\ load$ $[mg\ P/m^2 * a]$
$\left[P_c^{sp}\right]$ $= critical\ specifc\ Phosphorus\ concentration\ at\ the\ time\ of\ circulation\ in\ spring\ time$ $[mg/m^3]$
q_s $= hydraulic\ load$ $[m/a]$
$\bar{z}$ $= mean\ depth$ $[m]$

L_c $= critical\ specifc\ Phosphorus\ load$ $[mg\ P/m^2 * a]$
$\left[P_c^{sp}\right] = 20.....30\ \mu g/l$ $in\ the\ Matrimandir\ Lake$
$q_s = \frac{\bar{z}}{\tau_W}\ oder\ \frac{Q_i}{A_O}[m/a]$
τ_W $= retention\ time\ of\ the\ water\ in\ the\ lake$ $[a]$
Q_i $= annual\ face\ [m^3/a]$
A_O $= surface\ area\ of\ the\ lake$ $[m^2]$

q_s = 7,67 m / 0,18 a = 42,61 m/a

q_s = 8.000.000 m³/a / 186.000 m² = 43,03 m/a

$$L_c = 30\,mg/m^3 * 43{,}03\,m/a \left(1 + \sqrt{\frac{7{,}67m}{43{,}03\,m/a}}\right)$$

L_c = 1.835,89 mg/m²*a

The permissible and critical specific phosphorus load would be

$L_c = 341{,}48$ kg/a

The calculation in table 5.28 has been carried out for 5 lakes with different morphologies and for 3 different hydraulic loads, of 4 Mio. m³/a, 6 Mio. m³/a and 8 Mio. m³/a. From the summary in Table 5.28 and from Figure 5.2 it can be observed that the permissible Phosphorus load at different hydraulic loads is significantly lower in a smaller lake having a smaller mean depth and smaller surface.

Table 5.29: Pollution of Phosphorus of Stormwater Runoff in different Areas in Auroville
*Limnol. Report Poza Honda

	Area	P*	P
	m²	kg/ha*a	kg/a
Commercial Area	1,260,000	0,80	100,80
Public Green Area	1,040,000	0,05	5,20
Roads, Yards and Private Green areas	1,540,000	0,50	77,00
Agricultural Land	6,030,000	0,40	241,20
Forest Area	6,030,000	0,05	30,15
Lake Surface	186,000	0,20	3,72
Total	**16,086,000**		**458,07**

Table 5.30: Summary of critical Phosphorus Load of the Matrimandir Lake for optimal Morphological Parameters

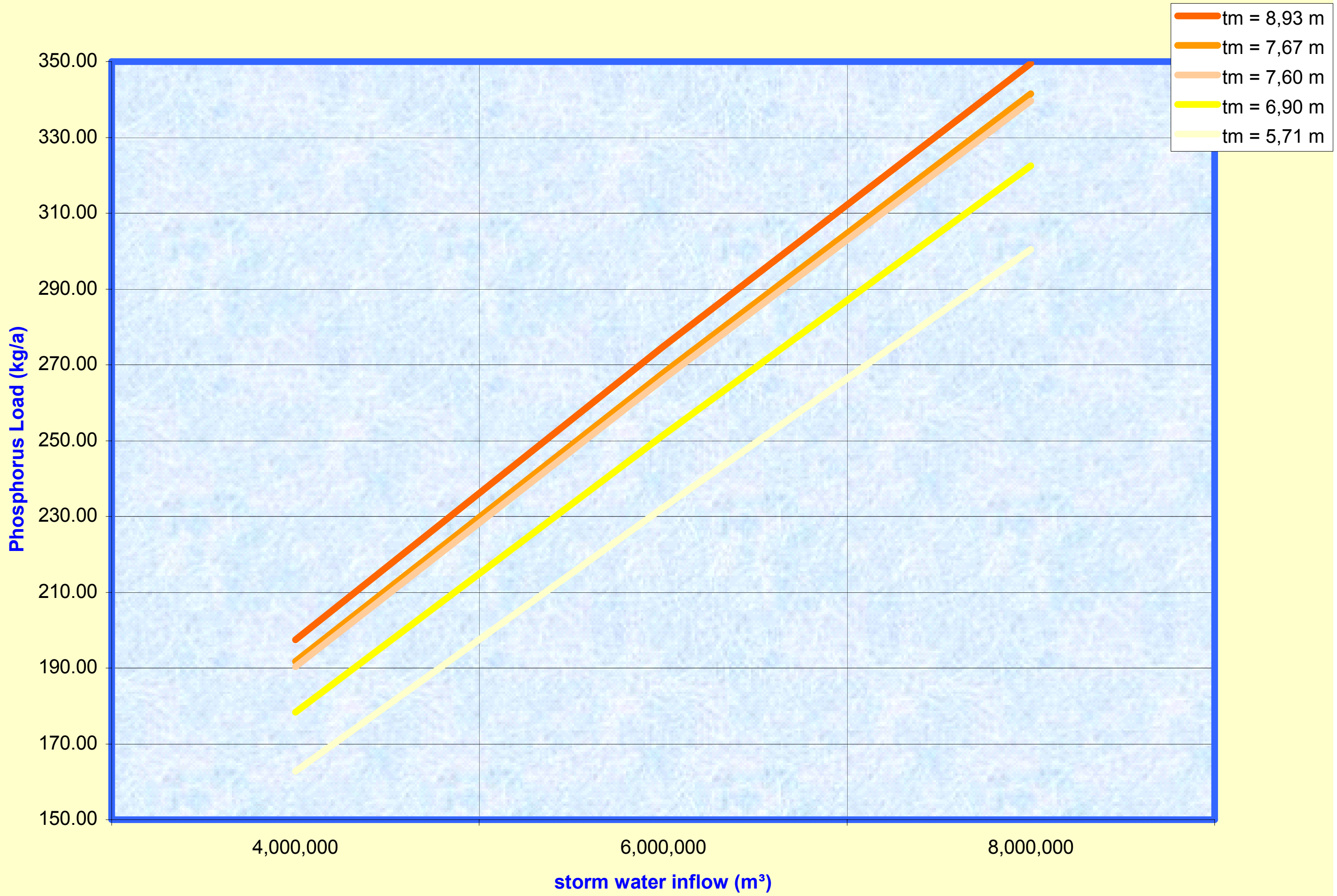
	Annual Inflow		
	m³/a	m³/a	m³/a
	4,000,000	**6,000,000**	**8,000,000**
t_m Mean Depth	**Critical Phosphorus Load of the Lake**		
m	**Kg/a**	**kg/a**	**kg/a**
8,93	197,42	274,84	**349,53**
7,67	191,73	267,87	341,48
7,60	190,40	266,23	339,57
6,90	178,32	251,43	322,47
5,71	162,79	232,39	300,49

Table 5.31 Estimate of the Critical Phosphorus Load for optional Morphological Parameters of the Matrimandir Lake

		Small Lake			Medium Lake			Large Lake		
Psp/c Critical Phosphorus Load at the Time of Circulation	mg/m³	30	30	30	30	30	30	30	30	30
Annual Inflow	m³	4.000.000	6.000.000	8.000.000	4.000.000	6.000.000	8.000.000	4.000.000	6.000.000	8.000.000
Lake Surface	m²	89.322	89.323	89.324	137.027	137.027	137.027	181.102	181.102	181.102
Retention Time	a	0,13	0,09	0,06	0,24	0,16	0,12	0,34	0,23	0,17
Lake Volume	m³	510.250	510.251	510.252	945.610	945.610	945.610	1.376.280	1.376.280	1.376.280
t_m Mean Depth	m	5,71	5,71	5,71	6,90	6,90	6,90	7,60	7,60	7,60
qs Hydraulic Load		44,76	67,14	89,52	29,19	43,78	58,38	22,09	33,13	44,18
Lc Critical Specific Phosphorus Load	mg/m²*a	1.822,49	2.601,71	3.364,01	1.301,37	1.834,86	2.353,34	1.051,35	1.470,04	1.875,01
Critical Phosphorus Load of the Lake	**kg/a**	**162,79**	**232,39**	**300,49**	**178,32**	**251,43**	**322,47**	**190,40**	**266,23**	**339,57**

		Large Lake			Large Lake		
Psp/c Critical Phosphorus Load at the Time of Circulation	mg/m³	30	30	30	30	30	30
Annual Inflow	m³	4.000.000	6.000.000	8.000.000	4.000.000	6.000.000	8.000.000
Lake Surface	m²	186.000	186.000	186.000	186.000	186.000	186.000
Retention Time	a	0,36	0,24	0,18	0,42	0,28	0,21
Lake Volume	m³	1.426.000	1.426.000	1.426.000	1.660.000	1.660.000	1.660.000
t_m Mean Depth	m	7,67	7,67	7,67	8,93	8,93	8,93
qs Hydraulic Load		21,51	32,27	43,03	21,52	32,28	43,04
Lc Critical Specific Phosphorus Load	mg/m²*a	1.030,82	1.440,15	1.835,89	1.061,40	1.477,64	1.879,20
Critical Phosphorus Load of the Lake	**kg/a**	**191,73**	**267,87**	**341,48**	**197,42**	**274,84**	**349,53**

Fig. 5.7: Critical Phosphorurs Load for optimal morphological Parameters of the Marimandir Lake



It can therefore be concluded that the surface of the lake and the mean depth has to be maximized in order to achieve a stable water body and to prevent its eutrophication.

It can be expected that the phosphorus load in the stormwater runoff will be between 458 P_{tot}/a and1.087 kg P_{tot}/a. If the lake is large and designed to accept a critical phosphorus load of 341,48 kg P_{tot}/a the pre-treatment prior to the discharge into the lake has to remove up to 745,5 kg P_{tot}/a, equal to 68,6 % of the total phosphorus load.

The efficiency for the removal of phosphorus through mechanical treatment can be expected to be 20 %.
From the total of 1,087 kg P_{tot}/a it can be expected that

217,4 kg P_{tot}/a

can be removed from the outflow of the stormwater storage tank during the 1st, 2nd and 3rd treatments.

From the remaining 869,6 kg P_{tot}/a, approximately 30 % or an equivalent amount of 260,9 kg P_{tot}/a can be removed through biological treatment (4th treatment) during re-circulation, leaving 608,7 kg P_{tot}/a in the water fed to the slow sand filtration.

Assuming a further removal of 30 % during the biological treatment in the slow sand filtration process, amounting to 182,61 kg P_{tot}/a, the remaining phosphorus load in the inflow of the Matrimandir Lake would be 426,09 kg P_{tot}/a, exceeding the critical phosphorus load of L_c = 341,48 kg P_{tot}/a by 19,8 %.

If the phosphorus removal in the re-circulation system could be increased to 45 %. (BERNHARDT, 1979), removing 391,32 kg P_{tot}/a from the water, then the load would be reduced to 478,28 kg P_{tot}/a. In that case the slow sand filtration could remove additionally 143,48 kg P_{tot}/a (30 %) from the flow, leaving a load of 334,8 kg P_{tot}/a in the inflow of the lake, which is then well below the critical phosphorus load of L_c = 341,48 kg P_{tot}/a.

It can be observed from this calculation that it is a rather difficult process to remove the phosphorus from the stormwater, and that a significant uncertainty remains, which is the quantity of phosphorus that will be collected from the runoff in the city and deposited into the storage tanks in the Green Belt.

Since the Matrimandir Lake remains the final and decisive stage in the transformation of the polluted stormwater runoff into the drinking quality water, and since this eco-system reacts very sensitively to phosphorus, it can only be recommended to make this lake as large and deep as possible so that its critical phosphorus loading capacity be increased accordingly, thereby ensuring that the lake remains mesotrophic with total phosphorus concentrations well below 30 µg P_{tot}/l.

Phosphorus can be removed through chemical precipitation or membrane filtration. These methods are very costly if applied to large volumes of water and therefore have not been considered in this study. The exclusive use of natural and biological treatment methods has been emphasized.

Several alternatives have been evaluated, and the following recommendations for the design of the lake can be given.

Since the Cuddalore Sandstone is approximately 30 m at the Matrimandir, it is recommended to limit the depth of the lake to 10 m. If the sealing of the lake bottom is achieved with the use of vacuum condensed clay, the slopes of the sides of the embankments should be n = 1 : 3. The bottom and embankments have to be covered with granulates and sand. On the outer edge of the Peace Area that borders the Matrimandir Island and the Lake, and in certain parts of the outer banks of the lake on the side of the city, the embankments can be built with concrete walls.

The outer circumference of the banks of the lake could be close to a circle with a radius of 300 m and 290 m. This would provide a minimum distance between the outer and inner banks of the lake of 90 m and a maximum distance of 160 m.

The lake surface will then have an area of

$$\underline{A_0 = 186.000\ m^2.}$$

and the lake volume will be

$$\underline{V = 1.426.000\ m^3}.$$

According to these morphological parameters the derived mean depth is

$$t_m = 7{,}67\ m.$$

From the surface of the lake approximately 306,000 m³/a of water will be evaporated. The precipitation on the lake surface will be 232,740 m³/a. About 19.600 m³/a will percolate through the sealing of the lake bottom if the permeability coefficient of the sealing is k_f = 10E-11. The net evaporation loss in an average year will be

73,300 m³/a; in a dry year the water loss will be 193.300 m³/a; and in a wet year there will be an overflow to the groundwater recharge of 162,700 m³/a. No runoff should be drained directly into the lake. The embankment of the lake towards the city and on the Matrimandir Island should be higher than the surrounding area and the surface gradient should not lead towards the lake, so no runoff can reach the lake water. The lake and its surrounding areas should remain the cleanest and the most protected location in the city.

If the size of the lake is reduced and an optimal morphology of the lake is not respected, the phosphorus load that the lake can safely handle would be significantly reduced and affected. As a result of any reduction in size of the lake and deviations from an optimal morphology, the content of phosphorus in the water that is to be fed into the lake would have to be reduced significantly to quantities that are lower than those indicated in this study. In such an instance much more effort would have to go into the removal of phosphorus before the stormwater is allowed to enter the lake. If excess loading of the lake with phosphorus is allowed it could result in the eutrophication of the lake, which in turn would render the water unfit for potable use and groundwater recharge, and it would therefore endanger the sustainable supply of water for the city. (See drawing No. 42.02/1.2.4, 1.2.5, 1.3.5).

5.5.3.12 Dimensioning of the Outflow Rapid Sand Filter at the Matrimandir Lake (8th Treatment)

The outflow from the Matrimandir Lake will be fed to the groundwater recharging facility that would be located on the island in the Matrimandir Gardens.
For the infiltration into the ground, infiltration trenches (rigole) filled with sand will be used. To prevent the clogging of the pipes which are distributing the clear water in the trenches as well as the sand fill in the trenches, the outflow of the lake has to be free of filterable solids. Therefore the use of a rapid sand filter as the last and 8th treatment, charged by gravity, will be proposed.

The location of the filter will be near the outflow - opposite the inflow - and the filtered water would have to be conveyed beneath the lake to the intake of the infiltration system located on the bank of the Matrimandir island. Since the depth of the infiltration system will be determined by the head losses through the outflow filter unit, the friction losses in the pipe system have to be minimized.

The maximum flow to the rapid sand filter has been estimated to be

$$Q = 8{,}000{,}000 \text{ m}^3/\text{a} = 913{,}24 \text{ m}^3/\text{h} = 254 \text{ l/s}.$$

This outflow from the lake requires a pipe diameter of

DN 700.

The velocity will be $v = 0{,}66$ m/s.

The rapid sand filter will have a filter of h = 1,000 mm thickness; the filter material will have a grain size of 0,8 mm up to 1,2 mm; and the filter velocity will be $v_F = 10$ m/h. The depth of the water above the filter shall not exceed 0,5 m.
The backwash (Q_R) of the filter requires 1 - 2 % of the filter water. The backwash has to be discharged into the stormwater drainage system.

The required surface of the filter can be estimated as follows:

$$
\begin{aligned}
Q &= 8{,}000{,}000 \text{ m}^3/\text{a} \\
Q &= 913{,}24 \text{ m}^3/\text{h} \\
Q_R &= 913{,}24 \text{ m}^3/\text{h} \times 0{,}02 = 18{,}26 \text{ m}^3/\text{h} \\
Q_{SF} &= 931{,}51 \text{ m}^3/\text{h} \\
v_F &= 10 \text{ m/h} \\
A_{SF} &= Q / v \\
A_{SF} &= 931{,}51 \text{ m}^3/\text{h} / 10 \text{ m/h} \\
A_{SF} &= 93{,}15 \text{ m}^2
\end{aligned}
$$

The filtration of the outflow requires 5 rapid sand filter units of B = 3 m width and L = 6,7 m length with a filter surface of $A_{SF} = 20$ m and a height of H = 3,5 m. The operation time can be estimated to be 20 to 100 hours and the time required for the combined air and water backwash at a velocity of 40 - 90 m/h can be estimated to be 15 to 20 minutes.
Considering the required time for backwash, maintenance and repairs for the filter it is recommended to construct an outflow filter of

6 units, B = 3 m, L = 6,7 m, H = 3,5 m, $A_F = 120$ m².

The height of the filter of H = 3,5 m will be determined by the filter bottom, including the filter chamber with the equipment for the backwash and to prevent the emptying of the filter. The freeboard of the filter has to be sufficient to allow the ventilation of the filter basin. The required filter surface has been estimated according to the infiltration capacity of the groundwater recharge system, and can be obtained from table 5.32.

Table 5.32 Dimensioning of the Rapid Sand Outflow Filter of the Matrimandir Lake

Outflow from Matrimandir Lake	Annual Discharge from Matrimandir Lake	Outflow from Matrimandir Lake		Required Filter Surface (filter velocity = 10m/h)
Alternatives	[m³/a]	[m³/h]	[l/s]	[m²]
Alternative 1	4.000.000	457	127	46.58
Alternative 2	6.000.000	685	190	69.86
Alternative 3	8.000.000	913	254	93.15

The head losses in the pipes of the inflow and outflow of the filter have been estimated for the different loads of 4 M m³/a, 6 M m³/a and 8 M m³/a and indicated in table 5.30. The difference of the water level in the lake and groundwater recharge facility will be 0,62 to 0,80 m. (see drawing 42.02/1.2.5).

Table 5.33 Estimate of the Head Losses in the Outflow Filter of the Matrimandir Lake

Outflow from Matrimandir Lake	Annual Discharge from Matrimandir Lake	Outflow from Matrimandir Lake		Head Loss in the Rapid Sand Filter	Outflow Pipe, Diameter DN	Lake Outflow Pipe, Head Losses for Pipe Length of 100 m	Filter Outflow Pipe Head Losses for Pipe Length of 200 m	Total Head Loss
Alternatives	[m³/a]	[m³/h]	[l/s]	[mm]	[mm]	[mm]	[mm]	[mm]
Alternative 1	4.000.000	457	127	560	600	20	40	620
Alternative 2	6.000.000	685	190	560	700	50	100	710
Alternative 3	8.000.000	913	254	560	700	80	160	800

5.5.4 Groundwater Recharge

The overflow from the Matrimandir Lake will be fed to slow sand filters by gravity, where most of the filterable solids will be removed. The entire annual overflow up to 8,000,000 m³/a will be infiltrated into the ground within the Matrimandir Island or at the outer circumference of the Lake. The Island is the highest, cleanest and most protected site in the city.

The stormwater runoff and the wastewater from the island have to be drained beneath the lake, preferably in a service tunnel towards the city. All solid waste or organic waste should be removed from the island so that no pollution can enter the recharged groundwater. For the recharge of the groundwater an interconnected system of underground trenches (rigole) has been proposed.

The depth of the trenches can vary from H = 2,00 m to H = 3,00 m, with a width of at least B = 1,0 m. On the bottom of the trenches a slotted pipe should be used for the uniform distribution of the water with DN 150 mm to DN 300 mm. Manholes at a distance of 60 m to 80 m should be provided to clean the various sectors or to allow the disconnection for repair or maintenance.
The location of the infiltration trenches should be next to or beneath the paths or lanes on the island. The network should be fed by gravity, to avoid additional pumping. The infiltration capacity of this system determines the required storage volume of the tanks in the GreenBelt and the retention time of the water in the lake. It determines as well the possibility to recharge the groundwater in years with high annual rainfall in order to store water in the ground that can be extracted in dry years.

The infiltration tests near the Matrimandir executed in 2002 obtained a very low infiltration rate of

$$k_f = 1{,}20 \text{ E-06 m/s to } 2{,}10 \text{ E –06 m/s}$$

at a depth of 1,70 m and 1,90 m.

The highest infiltration rate had been found at a depth of 1,53 m at

$$k_f = 5{,}77 \text{ E-05 m/s}$$

but it was noted that the soil was distributed in this area. At a distance of approximately 500 m from the Matrimandir the infiltration rate was measured at a depth of 1,25 m and was found to be

$$k_f = 1{,}36 \text{ E-05 m/s.}$$

Infiltration tests from the excavation pits of foundations near the Matrimandir showed higher rates than k_f = 1,20 E-06 m/s, therefore it can be concluded that an infiltration test should be carried out on an infiltration rigole with the proposed sizes in order to derive the exact infiltration rate, which then could be used to dimension the system.

Moreover, the double ring infiltrometer test may deliver very conservative and very low rates in this type of soil.

The required length of the infiltration trench system has been calculated for three infiltration rates of

k_{f1} = 2,0 E-06 m/s (on the island)
k_{f2} = 5,0 E-06 m/s (outside the lake)
k_{f3} = 1,0 E-05 m/s (on the island)

The infiltration system has to be designed for an infiltration capacity required in dry, mean and wet years at 4,000,000 m³/a, 6,000,000 m³/a and 8,000,000 m³/a respectively.

The infiltration rate of the infiltration trenches has been calculated on the basis of the following equations:

$$Q = A \times k_f$$
$$A = (b + 2 \times h) \times l$$

Q = infiltration rate
k_f = coefficient of permeability
A = wetted perimeter
B = bottom width of the infiltration trench (rigole)
h = utilizable depth of the infiltration trench
l = length of the infiltration trench

This equation differs from the common equation used in the literature of:

$$Q = A \times k_f / 2$$

In this case the infiltration trench will not be used by occasional charging, it is proposed to feed the system continuously without interruption. As it can be expected that the soil surrounding the infiltration trench will be saturated, the unlimited permeability of the saturated soil can be applied.

The results of the calculation can be obtained from table 5.31. Using the lowest infiltration rates of the soil (k_{f1}), the required length of the infiltration trench of Alternative I will be:

L_1 = 12,000 m (Matrimandir Island and 2 x surrounding the Matrimandir Lake).

Using the lowest and higher infiltration rates of the soil (k_{f1} and k_{f2}), the required length of the infiltration trench of Alternative II will be:

L2 = 10,200 m (Matrimandir Island and 1,33 x surrounding the Matrimandir Lake).

On the basis of the higher infiltration rate of the soil (k_{f2}) of Alternative III the length of the infiltration system can be reduced up to:

L3 = 9,000 m (Matrimandir Island and 1 x surrounding the Matrimandir Lake).

The most economic length of the infiltration trench can be obtained from Alternative IV if the highest infiltration rate (k_{f3}) is used. The length of infiltration system can then reduced to

L_4 = 5,100 m

Then the system can remain entirely on the Matrimandir Island (see drawing 42.02/1.2.5, Annex 1.7 and 2.6)

Table 5.34 Dimensioning of the Infiltration Trench System of the Matrimandir Lake

	Location of Infiltration Trench	Permeability	Width of the Infiltration Trench	Depth of the Infiltration Trench	Infiltration Rate	Infiltration Rate	Infiltration Rate	Required Length	Infiltration Capacity	Infiltration Capacity
Alternatives		k_f [m/h]	b [m]	h [m]	Q_s [l/s x m]	Q_s [m³/d x m]	Q_s [m³/month x m]	L [m]	Q_s [m³/month]	Q_s [m³/year]
Alternative I	MM Island	2,0E-06	1,0	2,0	1,0E-02	0,86	25,92	6.200	160.704	1.930.000
	City/Lake Edge	2,0E-06	1,0	3,0	1,0E-02	1,21	36,29	6.000	217.728	2.610.000
Total									**378.432**	**4.540.000**
Alternative II	MM Island	2,0E-06	1,0	2,0	1,0E-02	0,86	25,92	6.200	160.704	1.930.000
	City/Lake Edge	5,0E-06	1,0	3,0	1,0E-02	3,02	90,72	4.000	362.880	4.350.000
Total									**523.584**	**6.280.000**
Alternative III	MM Island	5,0E-06	1,0	2,0	1,0E-02	2,16	64,80	6.000	388.800	4.670.000
	City/Lake Edge	5,0E-06	1,0	3,0	1,0E-02	3,02	90,72	3.000	272.160	3.270.000
Total									**660.960**	**7.940.000**
Alternative IV	MM Island	1,0E-06	1,0	2,0	1,0E-02	4,32	129,60	5.1000	660.960	7.930.000
	City/Lake Edge	1,0E-06	1,0	3,0	1,0E-02	6,05	181,44	0	0	0
Total									**660.960**	**7.930.000**

5.6 Limitations and Further Research

5.6.1 Hydrology

Infiltration Rate in the Peace Area

The infiltration tests executed by SCHLENTHER in 2002 as infiltrometer tests could not provide reliable and safe data. It is therefore recommended to execute an on-site test using 5 m to 10 m of infiltration trench in the dimensions proposed for the execution.

Transmissivity of the Cuddalore Sandstone

As part of the ongoing hydrological study of the Auroville area, the transmissivity of Cuddalore Sandstone has to evaluated, using well tests. Only then can the retention time of the recharged groundwater be determined.

Groundwater Model

The existing hydrogeological data have to be utilised to prepare a groundwater model for the groundwater flow in the Cuddalore Sandstone beneath Auroville. It has to be evaluated where the main groundwater movement from the infiltration zone towards the sea will take place.

The following questions have to be answered:

a) Where will be the best locations for the extraction wells?
b) How much water can be stored in the Cuddalore Sandstone Aquifer? and
c) What will be the retention time of groundwater within the Auroville limits?

5.6.2 Water Rights

When the Auroville territory is fully afforested and bunded and soil and water conservation measures are completed, then the runoff from the hill is minimized. This will be the best time to define the water rights for the low lying areas, prior to the construction of the city. If it is possible to measure the discharge from Auroville, especially from the Irumbai and Alankuppam Tanks, this water could be guaranteed to those using that water in the further development of the Auroville project.

As soon as the construction of roads and buildings proceed, the runoff will increase. The runoff from the city that is generated due to the sealing of the surface is essential for the water supply of the city. Therefore, it has to be harvested and infiltrated.

If the system is in full operation, sufficient groundwater and possibly even treated wastewater will be available also for the neighbouring environments.

5.6.3 Watershed Management

There is already a competition for water in the region and a search for the development of new resources. There is also well the increasing danger of depletion and salinisation of groundwater resources in the area. Auroville has a planned population of 50,000 inhabitants that will need to migrate to this area. This will add further stress on the existing resources available to the present population, especially if Auroville fails to manage its own water resources in an optimal manner.

To minimize the potential conflicts between Auroville and its neighbours, it is therefore strongly recommended to extend the ongoing programme of water conservation, tank rehabilitation and afforestation to improve the prevailing water management and increase the water availability in the region.

5.6.4 Soil Management

The construction of a new city makes a substantial demand on building materials. Soil is required for landscaping, and the construction of roads and buildings. The red soil in Auroville is of a particular value in the region. It should be noted that the soil obtained from the excavation for the construction of structures, foundations, pipe networks, the water courses and ponds in the parks, the tanks in the GreenBelt and the Matrimandir Lake will provide 3 to 4 million m^3 of soil (which would cover the entire surface of Auroville by 15 cm to 20 cm!).

The excavation of the Matrimandir Lake will supply 1,5 million m^3 of soil (which would cover Auroville with 7,6 cm of soil). It is obvious from these figures that a soil management program is required and a model would have to be developed to determine the mass movements of soil and the required surface levels of the entire area for further development.

The excavation of the Matrimandir Lake has to be executed prior to the construction of the major parts of the city in order to minimize the impact on the residents of the township and also to make available soil at the earliest stage of construction and landscaping.

5.6.5 Construction of the Matrimandir Lake

The construction of the lake should be undertaken as early as possible. For the excavation a professional company should be employed. For the sealing of the lake vacuum condensed clay should be used (10 to 12 cm thickness). **The clay to be used for the sealing has not been identified in the region yet.**

For the first filling of the lake, groundwater has to be used. If the lake is filled within one year, wells drawing from the limestone aquifer with a total capacity of 45 l/s have to be used.

Once the lake is filled, the annual losses for evaporation and infiltration of 92,900 m^3/a in an average year (2,9 l/s) can be substituted by harvested and treated stormwater or from groundwater until the city has extended and the required runoff can be delivered.

5.6.6 Re-circulation System

The water courses and ponds in the four parks of the city and the GreenBelt have not been designed yet. At present the water courses can be obtained only from the model of Auroville.

The design of the re-circulation system in this report in based on the images of the Auroville model and therefore cannot be accurate. As soon as proposals for the water courses and ponds are presented, the estimates and the design of the re-circulation system have to be revised accordingly.

5.7 Estimated Costs

5.7.1 Estimated Costs for the Stormwater Management System

The estimated costs are based generally on unit prices available from India. In case these prices are not available, unit prices from Germany have been used. The following exchange rate has been used

1 Euro = 50 RS.

The costs for the construction of the stormwater harvesting in cisterns require the largest investment. The costs have been estimated to be

2,315,270,000 RS.

The construction costs for the stormwater drainage system have been estimated to be 59,267,000 RS.

The construction costs for the sedimentation basins and the storage tanks in the GreenBelt have been estimated to be

849,560,000 RS.

The construction costs for the filter pipe and the booster pump system, to lift the stormwater from the GreenBelt into the re-circulation system have been estimated to be

82,180,000 RS.

The construction costs for the re-circulation system, the filters, pumps and pipelines to lift the water into the Matrimandir Lake have been estimated to be

99,058,000 RS.

The construction costs for the Matrimandir Lake have been estimated to be

350,280,000 RS.

The costs for the filter and system for the groundwater recharge on the Matrimandir Island have been estimated to be

44,627,000 RS.

The total construction costs of the stormwater management system have been estimated to be

3,800,242,000 RS.

Detailed cost estimates are presented in table 5.35 and Annex 2.10.1 – 2.10.6.

5.7.2 Estimated Costs for Operation and Maintenance of Stormwater Management System

The annual costs for operation and maintenance of the domestic cistern system have been estimated to be 57,882,000 RS/a.

The annual costs for the operation and maintenance of the stormwater drainage system have been estimated to be

4,524,000 RS/a.

The annual costs for the operation and maintenance of the sedimentation basin and storage tanks in GreenBelt have been estimated to be

33,982,000 RS.

The annual costs for the operation and maintenance of the filter pipe and the booster pump system, to lift the stormwater from the GreenBelt into the re-circulation have been estimated to be

6,164,000 RS.

The annual costs for the operation and maintenance of the re-circulation system, the filters, pumps and pipelines to lift the water into the Matrimandir Lake have been estimated to be

7,429,000 RS.

The annual costs for the operation and maintenance of the Matrimandir Lake and the groundwater recharge on the Matrimandir Island have been estimated to be

29,618,000 RS.

The annual costs for the operation and maintenance of the stormwater management system have been estimated to be

139,598,000 RS.

The detailed cost estimates are presented in table 5.36.

5.7.3 Estimated Costs for the entire Water Management Scheme

The total costs for the entire water management scheme have been estimated to be

4,793,930,000 RS.

The detailed cost estimates are presented in table 5.35.
The costs related on the groundwater recharge are presented in table 5.37.

Table 5.35 Summary Estimate of Construction Costs

NO.	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	Total Costs [RS]	Total Costs * [€]
1	DRINKING WATER SUPPLY		
1.1	CONSTRUCTION OF DRINKING WATER EXTRACTION AND TREATMENT	103,111,000	2,062,000
1.2	CONSTRUCTION OF PIPE NETWORK FOR DRINKING WATER SUPPLY	91,869,000	1,837,000
1.3	CONSTRUCTION OF PIPE NETWORK FOR PROCESS AND IRRIGATION WATER SUPPLY	131,426,000	2,629,000
1	SUBTOTAL	326,406,000	6,528,000
2	STORMWATER MANAGEMENT		
2.1	CONSTRUCTION OF THE DOMESTIC CISTERN SYSTEM	2,315,270,000	46,305,000
2.2	CONSTRUCTION OF THE STORMWATER DRAINAGE SYSTEM	59,267,000	1,185,000
2.3	CONSTRUCTION OF THE STORMWATER RUNOFF SEDIMENTATION BASIN AND STORAGE TANKS IN THE GREEN BELT	849,560,000	16,991,000
2.4	CONSTRUCTION OF THE RAINWATER FILTRATION AND CONVEYANCE FROM THE GREEN BELT TO THE CITY	82,180,000	1,644,000
2.5	CONSTRUCTION OF THE RAINWATER RECIRCULATION AND FILTRATION IN PUBLIC PARKS	99,058,000	1,981,000
2.6	CONSTRUCTION OF THE MATRIMANDIR LAKE AND THE GROUNDWATER RECHARGE	394,907,000	7,898,000
2	SUBTOTAL	3,800,242,000	76,005,000
3	WASTEWATER MANAGEMENT		
3.1	CONSTRUCTION OF THE AUROVILLE SEWER NETWORK	189,552,000	3,791,000
3.2	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT EAST	121,215,000	2,424,000
3.3	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT WEST	356,515,000	7.130.000
3	SUBTOTAL	667,282,000	13,346,000
	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	4,793,930,000	95,879,000

* ... exchange rate 1€ = 50 RS

Table 5.36 Costs of Operation and Maintenance of the Stormwater Management

COSTS OF OPERATION AND MAINTENANCE OF THE STORMWATER MANAGEMENT		
No.	**Description**	**Total Annual Costs of O&M**
[-]	**[-]**	**[RS/a]**
1	DOMESTIC CISTERN SYSTEM	57,881,750
2	STORMWATER DRAINAGE SYSTEM	4,523,893
3	STORMWATER RUNOFF SEDIMENTATION BASIN AND STORAGE TANKS IN THE GREEN BELT	33,982,400
4	RAINWATER FILTRATION AND CONVEYANCE FROM THE GREEN BELT TO THE CITY	6,163,500
5	RAINWATER RECIRCULATION AND FILTRATION IN PUBLIC PARKS	7,429,350
6	MATRIMANDIR LAKE AND THE GROUNDWATER RECHARGE	29,618,025
TOTAL COSTS FOR OPERATION AND MAINTENANCE OF THE STORMWATER MANAGEMENT		**139,598,918**

Table 5.37 Estimate of the Costs for Water Supply

	Description	Construction Costs	Costs for M&O	Drinking Water[1] Costs / m^3		Process Water[2] Costs / m^3		Drinking and Process Water[3] Costs / m^3		Irrigation Water[4] Costs / m^3		Wastewater[5] Costs / m^3	
No.	**Costs for the Sanitary Infrastructure of Auroville**	[RS]	[RS/a]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]	[RS/m^3]	[€/m^3]
1	**WATER SUPPLY ***												
1.1	Drinking Water Extraction and Treatment	103.111.000	14.009.000	4,38	0,09			2,92	0,06				
1.2	Pipe Network for Drinking Water Supply	91.869.000	5.912.100	2,11	0,04			1,41	0,03				
1.2	Pipe Network for Process and Irrigation Water Supply	131.426.000	4.440.800							3,61	0,07		
1	**SUBTOTAL**	**326.406.000**	**24.361.900**	**6,49**	**0,1**	**0,0**	**0,0**	**4,33**	**0,09**	**3,61**	**0,07**	**0,00**	**0,00**
2	**STORMWATER MANAGEMENT ****												
2.1	Domestic Cistern System	2.315.270.000	57.881.750			85,2	1,70	28,40	0,57				
2.2	Stormwater Drainage System	59.267.000	4.523.893	1,39	0,03			0,93	0,02				
2.3	Stormwater Runoff Sedimentation Basins and Storage Tanks in the GreenBelt	849.560.000	33.982.400	11,58	0,23			7,72	0,15				
2.4	Rainwater Filtration and Conveyance from the GreenBelt to the City	82.180.000	6.163.500	1,90	0,04			1,27	0,03				
2.5	Rainwater Circulation and Filtration in Public Parks	99.058.000	7.429.350	2,30	0,05			1,53	0,03				
2.6	Matrimandir Lake and Groundwater Recharge	394.907.000	29.618.025	9,15	0,18			6,10	0,12				
2	**SUBTOTAL**	**3.800.242.000**	**139.598.918**	**26,33**	**0,53**	**85,2**	**1,70**	**45,95**	**0,92**	**0,00**	**0,00**	**0,00**	**0,00**
3	**WASTEWATER MANAGEMENT**												
3.1	Sewer Lines**	189.552.000	636.173									0,69	0,01
3.2	Wastewater Treatment Plant East*	121.215.000	9.091.125									3,14	0,06
3.3	Wastewater Treatment Plant West*	356.515.000	26.738.625									9,23	0,18
3	**SUBTOTAL**	**667.282.000**	**36.465.923**	**0,00**	**0,00**	**0,0**	**0,00**	**52,05**	**1,04**	**0,00**	**0,00**	**13,06**	**0,26**
	TOTAL COSTS FOR WATER SUPPLY	**4.793.930.000**	**200.426.741**	**32,82**	**0,66**	**85,2**	**1,70**	**102,33**	**2,05**	**3,61**	**0,07**	**13,06**	**0,26**

*... Time of depreciation 50 years
**... Time of depreciation 100 years

1)... with 3.668.250,00 m^3/a
2)... with 1.222.750,00 m^3/a
3)... with 2.445.500,00 m^3/a for Drinking Water and 1222750 m^3/a for Process Water
4)... with 1.956.400,00 m^3/a
5)... with 3.668.250,00 m^3/a

6 PRE-FEASIBILITY STUDY FOR THE WASTEWATER MANAGEMENT OF THE CITY OF AUROVILLE

6.1 Introduction

Recycling of water through the treatment and re-use of wastewater can make a positive contribution to the sustainability of available water resources. Central to the approach of water recycling is the concept of the utility of water whereby the water used is of a quality commensurate with its application. This then permits the exploitation of large water resources that are not necessarily of the highest purity. Moreover, domestic sewage carries substantial amounts of valuable nutrients among which nitrogen (N) and phosphorous (P) are dominant. Therefore a nutrient-rich treated wastewater can be conveniently used for irrigation purposes.

6.2 Existing Wastewater Management

At present there are several different wastewater treatment systems in operation.
Individual houses usually treat their wastewater with a septic tank and dispose of it through a soak pit. Larger communities provide septic tanks or Imhoff tanks as first stage, and root zone treatment plants or ponds for secondary treatment. In the Industrial Zone the first common effluent treatment plant (CEPT) is proposed.
In general, at present the treatment of most of the wastewater is not up to the Indian Standards and percolates into the first aquifer.

6.3 Proposed Wastewater Management Systems

6.3.1 Objectives

The primary objective of the treatment and re-use of wastewater as well as storm-water is to develop a water resource management system to ensure and secure the availability of freshwater so that the water demand at Auroville is sustainable met. Over the coming decades, the management of water resources will become one of the most important issues across industrialized nations as water availability and quality are likely to decrease. Given the already existing water problems encountered at Auroville, it is imperative that all sources of freshwater be considered and if possible tapped for ensuring a safe and secure supply.

Given the scarcity of freshwater at Auroville and the surrounding regions, it is proposed to obtain additional sources of freshwater by 1) treating wastewater to a degree that allows it to be re-used, at least for irrigation, and 2) harvesting all of the rainwater that falls on all the paved surfaces. This is described in detail below. An alternate and reliable source of freshwater is sought through the treatment and re-use of wastewater and the harvesting of rainwater for direct use and groundwater enrichment.

In addition to solving the problems of freshwater supply, this proposal also seeks to implement systems and technologies that are suitable and appropriate for the prevailing physical, environmental, social and economic conditions at the project site. Thus concepts for systems and technologies that adhere to the principles of resource optimization and sustainable development have been proposed here. In particular it has been ensured that the proposed systems and technologies meet the following criteria:

- Minimum dependency on complex infra-structure services,
- High self-sufficiency in respect to operation and maintenance of systems,
- Low vulnerability to destruction,
- Can accommodate significant variations in hydraulic and pollution loads without significant loss of efficiency,
- Can handle a large variety of pollutants present in today's domestic wastewater,
- High efficiency in treatment of wastewater – up to tertiary treatment and removal of pathogens,
- No, or limited, use of mechanical parts (except for the minimum use of pumps for the required lifting of wastewater and sludge),
- Use of simple hardware.
- Minimized inputs of energy,
- No use of chemicals for the treatment process,
- No requirement of skilled manpower,
- Low long-term capital, operating and maintenance costs,
- Applicable at any site and on any scale.
- Allows phasing of systems,

- Can be easily and cost-effectively expanded to accommodate increased loads,
- Simple construction,
- Use of appropriate materials.
- Use of indigenous materials and building technologies to the maximum extent,
- Reduction of sludge production (in the rootzone treatment process no sludge is generated, therefore the sludge handling and disposal problem is restricted only to primary and secondary sludge).
- Allows recovery and re-use of useful by-products also, at or near the site (e.g. fertilizer and compost),
- Allows re-cycling and safe re-use of wastewater,
- Achieves conversion of wastes into re-usable high quality by-products,
- Allows complete utilization of all possible waste resources,
- Ensures a proper final destination for any type of residues.
- Long life span of system,
- Large re-use of materials when system is decommissioned,
- Prevents environmental pollution problems, in particular pollution of air, water and soil,
- Ensures environmental protection,
- Enhances or maintains the quality of the surrounding environment (e.g. root zone treatment systems enhance bio-diversity by creation of a wetland ecosystem),
- Takes into account high public participation and acceptability to all social players.

6.3.2 Description of the Drainage Area

6.3.2.1 Location

The drainage area comprises the built-up area of the city of Auroville, which is a perfect circular area with a surface area of 4,9 km² and a diameter of 2,5 km on the top of a hill with the highest point at its centre

The built-up area is separated by four public parks into 4 zones, the Residential Zone, the International Zone, the Industrial Zone and the Cultural Zone.

The outer limit of the city area is determined by the Outer Ring-Road at a radius of 1,250 m from the centre and the Crown Road (the Inner Ring Road) located at a radius of 700 m from the centre separates the 4 zones from the city centre, which has the Peace Area with the Matrimandir in its middle. Each zone is framed by a Radial Road that acts as a main access road.

6.3.2.2 Topography

The city centre is located just next to the top of the hill that has an elevation of 54 m above mean sea level (MSL).

From the centre, the surface slopes down to the elevation of 43m above mean or MSL in the North, to 34m in the West, to 46m in the East and 50 m to the South. There is a main watershed from the North-East to South-West splitting the city area into two major catchment areas. There is a slope of 0,2 % to 0,7 % which allows the area to be drained comfortably by gravity.

6.3.2.3 Land Use

According to the Master Plan, the proposed land use of the Auroville Area with its five zones and the GreenBelt is as in table 6.1

Table 6.1: Proposed Land Uses by Zone – 2025 (City Area / Developed Area)

Use Zones	Area in ha	%	Principal Uses
Peace Area	**28,00**	**5,70**	Matrimandir, Lake, Gardens
1. Residential Zone	**173,00**	**35,20**	
Primary Residential	160,000	32,60	Residential houses, apartments in five sectors at different densities and basic Community Facilities
Crown	23,00	2,60	Shopping, Utilities, Communication, Recreation and Community Facilities of higher orders, Supporting Residential use.
2. International Zone	**68,00**	**13,90**	
Pavilions	63,50	12,90	National and International Pavilions, Conference and Exhibition Halls
Crowns	8,50	1,00	Utilities, Communication, Shops and other Common Facilities related to the main activity in the International Zone, including Housing and Staff Quarters

Use Zones	Area in ha	%	Principal Uses
3. Industrial Zone	**126.00**	**25,70**	
Economic	94,50	19,30	Non-polluting Manufacturing Units, including Cottage Industries
Crown	8,50	1,70	Hotels, Dormitories, Guest Houses and Supporting Facilities for the main activity in the zone.
Administration	7,00	1,40	Town Hall, City Administration Office and Housing
Vocational Training	16,00	3,30	Vocational Training Centres, Research, Institutions including Laboratories
4. Cultural Zone	**96,00**	**19,50**	
Major cultural	91,00	18,50	Educational Institutions, University, Sports Centres and Staff Quarters.
Crown	5,00	1,00	Shopping, Utility, Communication and Recreation Centres and related facilities supporting Cultural Activities in the zone including Housing.
Total	**491,00**	**100,00**	

Proposed Land Use in the Green Belt – 2025

	Area in ha	%	Principal Uses
Built (Existing settlements to be retained)	156	10.5	Auroville Communities and Village Residential Areas, Service Nodes and Utilities and Main Access Roads.
Unbuilt	1316	89.5	Farming and Forest type uses and Recreation, Bird and Wild Life
Total	**1472**	**100.00**	

The land use in the drainage area of the city area according to the Master Plan is defined as in Table 6.2.

Table 6.2: Detailed Land Use in City Area - 2025

Use	Extent in ha	%	Remarks
1. Residential	121	24.64	Residential Zone 80 % Other Zones 20%
2. Commercial	20	4.10	Mostly in Crown Area connecting in zones
3. Industrial	56	11.40	Industrial Zone / Manufacturing Units
4. Public & Semi-public	159	32.38	
a. Matrimandir	28	5.70	Peace Area
b. Pavilions	38	7.73	International Zone
c. Educational & Cultural	73	14.86	Cultural and Residential
d. Administration, utilities & other uses	20	4.07	Industrial and other zones
5. Open space & recreation	46	9.36	To be provided in all zones
6. Transport & Communication	89	18.12	To serve all zones
Total	**491**	**100.00**	

6.3.2.4 Road Network

According to the Master Plan the physical infrastructure is planned as follows:

Road Network: The road network, consisting of four types of roads, is planned to meet the future requirement of traffic and the functioning of the Township. The proposed road network and the hierarchy of roads is shown in the proposed land use plan as well as in Drawing No. 7. The road section is shown in Drawing 8. The four types of roads and access ways in order of hierarchy are as follows:

Access Roads to Auroville: Four principal accesses are proposed. Two from the Tindivanam-Pondicherry Road, connecting the Industrial Zone and the International Zone; two from the East Coast Road (ECR), linking the Residential Zone and the Cultural Zone. Thus each zone will have an independent access from state / national highways. These roads will provide a link to the outer ring road of the City.

There would be bypass links where the existing narrow roads pass through village settlements. The right of way of these roads is suggested to be 30 meters.

City ring roads: Two ring roads are proposed within the City area, one circumscribing the four main use zones and the other adjoining the utility zone which is designed as the Crown Road. The right of way of these is also suggested to be 30 meters. These two ring roads will help in distributing the traffic to the different zones.

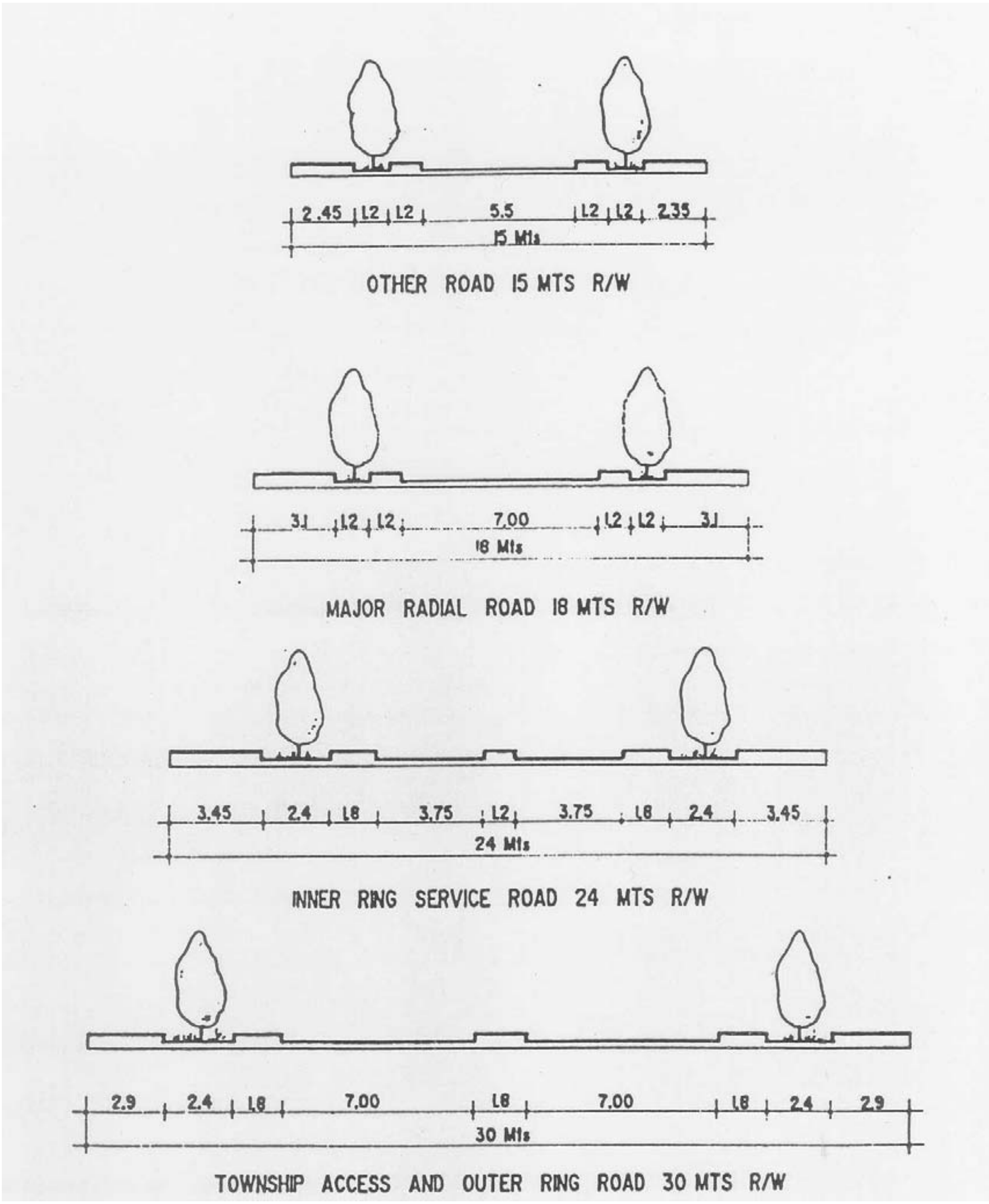
The entire City area has been envisaged as a “non-polluting vehicular zone”. Accordingly, the ring road circumscribing the City Area will be used progressively by non-polluting vehicles.

Internal Distribution Roads: The internal distribution roads consist of vehicular roads as well as pedestrian and cycle paths. The rights of way of vehicular roads would vary between 18 and 24 m depending upon their functions.

Service Nodes: Two kinds of service nodes are proposed. These service nodes are provided in the GreenBelt and are proposed at the intersection of the four main access roads linking the township and the City area. The first one would be called Primary Node and the latter, the Secondary Service Node, as indicated in the schematic layout of Service Nodes. These service nodes will provide adequate parking and trans-shipment space for changing over to “non-polluting” transport before entering the City. These service nodes will also offer other facilities for providing a convenient interface with neighbouring village settlements.

In addition to the main categories of roads discussed above, two bypass roads are also suggested, one in the north and the other in the south of the township to facilitate diversion of traffic which is not destined for Auroville.

Figure 6.1: Road Sections



In the Auroville Mobility Concept of Planungsbüro Billinger, Stuttgart 2001, a modification of the traffic and road concept was proposed as shown in Figs. 6.2, 6.3 and 6.4.

The aim of the Mobility Concept was "...to work out the mobility parameters of the Master Plan with more details. Based on the Master Plan's general considerations on traffic, especially the aim to giving preference to non-polluting movement, a network of roads and pathways has been proposed. A shuttle bus system is recommended to complete the network, connected to the service nodes specially developed for Auroville. In conclusion, some recommendations have been given as to how a motor-free city can be realized in carefully chosen steps."

Table 6.3: Auroville Town Plan – Basic Distances

Location	Length and Distances		Walking Time*
		Metres	Minutes
Crown Road	Radius	700	10
	Diameter	1400	20
	Circumference	4400	63
Outer Ring Road	Radius	1250	18
	Diameter	2500	36
	Circumference	8000	114
Green Belt Limit	Radius	2500	36
	Diameter	5000	71
	Circular	16000	228
Peace Area – Crown Road		350	5
Crown Road – Outer Ring Road		550	8
Outer Ring Road – Green Belt Limit		1280	18
Outer Ring Road Diagonal		2800	36
Green Belt Limit Diagonal		5000	71

*In metres and minutes walking time Speed of walking: 70 m per min.

Figure 6.2: Auroville Township dimensions (Source: Auroville Mobility Concept)

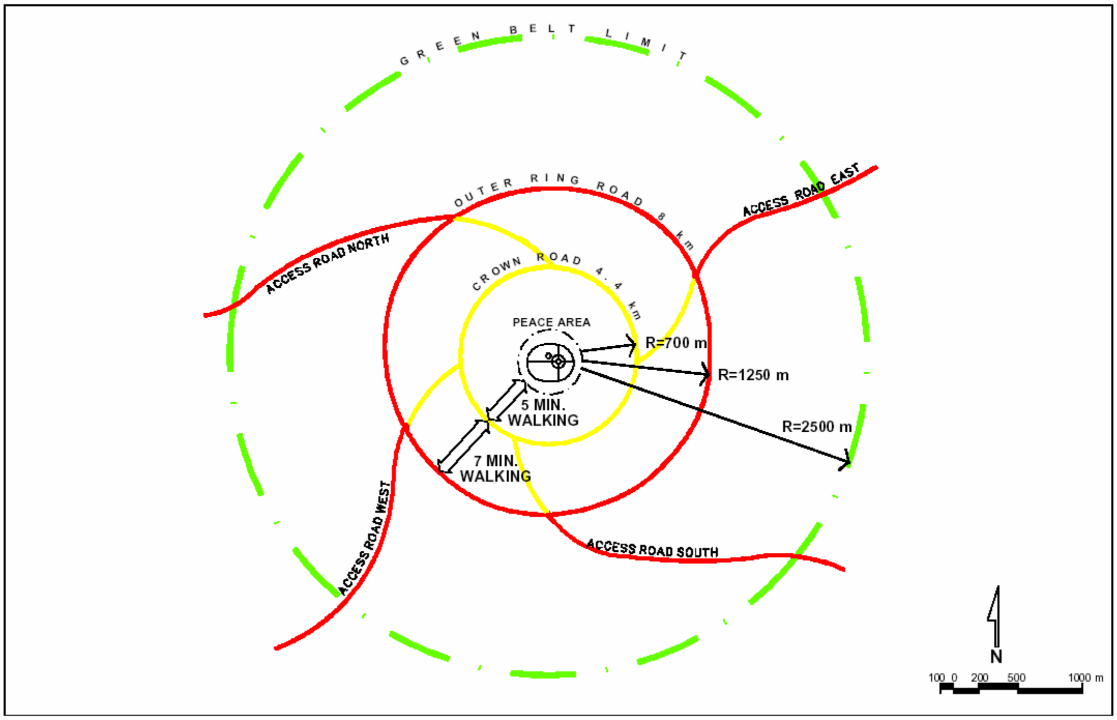


Figure 6.3: General Mobility Pattern (Source: Auroville Mobility Concept)

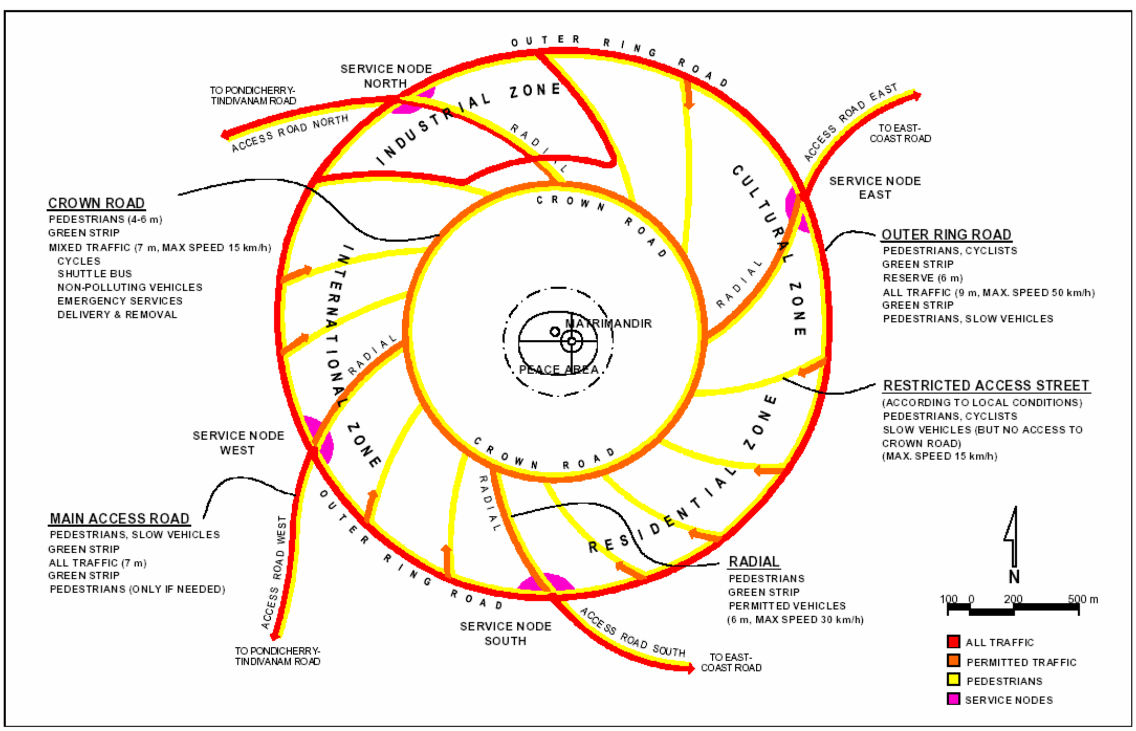
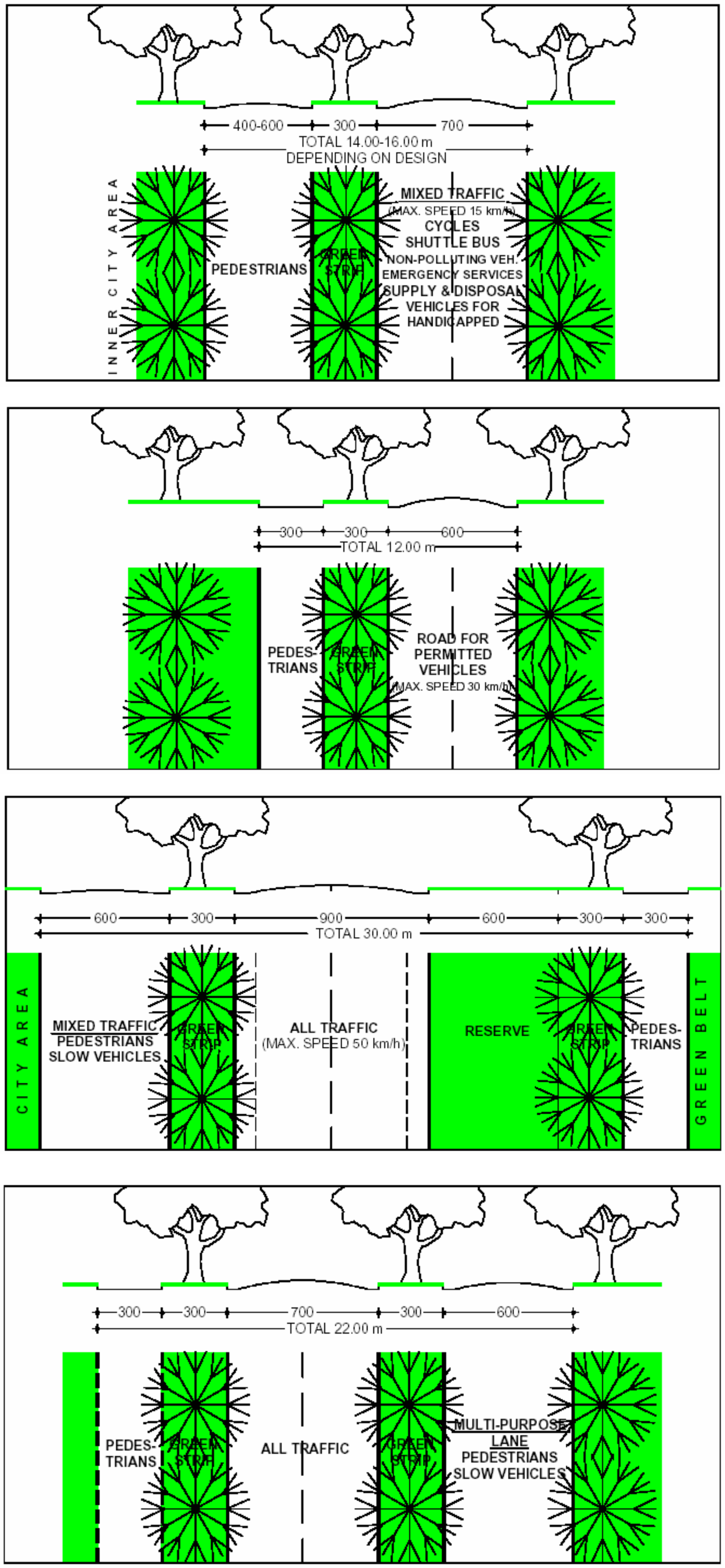


Figure 6.4: Section of Roads (Source: Auroville Mobility Concept)



6.3.2.5 Drainage Areas

According to the Master Plan the five zones of the City have the following functions:

Residential Zon	-	housing area
International Zone	-	Visitor Centres National Pavilions
Industrial Zone	-	industry and commerce
Cultural Zone	-	sports and culture
City Centre	-	administration and housing areas

The drainage area of each zone is estimated to be:

Table 6.4: Drainage Area

Residential Zone	160 ha
International Zone	63 ha
Industrial Zone	95 ha
Cultural Zone	91 ha
City Centre	82 ha
Total drainage area	491 ha

For these five zones wastewater can be drained from three catchment areas:

Table 6.5: Location

Northern Drainage Area	333 ha
South West Drainage Area	63 ha
South East Drainage Area	95 ha
Total of drainage area	491 ha

6.3.2.6 Population

According to the Master Plan Auroville consists of 6 Zones each generating different types of wastewater:

Residential Zone	40,000 inhabitants
International Zone	600 inhabitants
Industrial Zone	1,800 inhabitants
Cultural Zone	600 inhabitants
City Centre	5,000 inhabitants
Green Belt	2,000 inhabitants
Total	50,000 inhabitants

Additional generation of wastewater results from external users the Cultural and International Zones and from visitors.
The wastewater generation of commercial organizations, such as hotels, restaurants and other non-commercial activities such as cultural, sporting and other events and functions, also have to be considered.

The specific generation of wastewater from the Industrial Zone has to be estimated from the proposed industries:

Residential Zone	500 PE (Population Equivalent)
International Zone	1,500 PE (Population Equivalent)
Industrial Zone	10,000 PE (Population Equivalent)
Cultural Zone	3,500 PE (Population Equivalent)
City Centre	1,500 PE (Population Equivalent)
Total	17,000 PE (Population Equivalent)

The generation of wastewater of Auroville has to be estimated for a population of 50,000 and 17,000 PE (Population Equivalent) amounting to 67,000 PE.

6.3.3 Dry Weather Flow

The daily generation of wastewater is estimated to be:

$$w = 150 \text{ l/capita} \cdot \text{day}$$

The peak flow is calculated on the basis of:

h = 12 hours

For the city of Auroville the domestic dry weather flow rate can be estimated to be:

Q_{t_D} = 50,000 PE x 150 l/PE·d / 3,6
Q_{t_D} = 86,8 l/s

The peak flow rate can be estimated to be:

$Q_{\max_D}$ = 50,000 PE x 150 l/PE·d / 3,6 x 12 h = 173,6 l/s

$Q_{\max_D}$ = 173,6 l/s

The industrial / commercial wastewater flow rate can be estimated to be:

Q_{t_G} = 17,000 PE x 150 l/PE·d / 3,6 x 24 h = 29,5 l/s
Q_{t_G} = 29,5 l/s

The infiltration into the sewer system can be estimated to be:

Q_f = (86,8 l/s + 29,5 l/s) x 0,3 = 34,9 l/s
Q_f = 34,9 l/s

The average combined flow of domestic and industrial / commercial wastewater flow is estimated to be:

Q_t = 86,8 l/s + 29,5 l/s = 116,3 l/s
Q_t = 116,3 l/s

The peak dry weather flow can be estimated to be:

$Q_{\max}$ = 173,6 l/s + 59,0 l/s + 34,9 l/s = 267,5 l/s

$Q_{\max}$ = 267,5 l/s

During the rainy season an infiltration of 30 % will be added to the flow:

w_f = 50 l/capita·day

6.4 Proposed Drainage System

6.4.1 Method of Drainage

For the proposed re-use of stormwater runoff and sewage, two separate drainage systems are required. It is proposed that the stormwater is drained in open collectors. The sewage has to be drained in a closed system, using stoneware pipes.
The topography of the city of Auroville shows one major watershed and permits the design of the sewer system for only two catchment areas, entirely based on gravity flow. The largest catchment area drains by gravity to the north-west and the smaller catchment area drains to the east.

Domestic wastewater can be treated in order to make it suitable for re-use by way of conventional treatment technologies. Since the industries located in the Industrial Zone will continuously change the nature and volumes of production during the course of time, the properties of the wastewater produced there will vary accordingly. It is therefore proposed to provide the Industrial Zone with a separate drainage system as well as a separate treatment plant. Thus the city will be in a position in the future to adopt adequate treatment processes that can adapt to the changing properties of the wastewater,r based on whether the treated effluent is fit for reuse or if it has to be appropriately disposed off.
Due to the favourable topography and gradients, all the sewers can be laid with sufficient slope, so that the entire area can be drained by gravity to two locations. The sewers have to be laid in the roads and the main collectors will be located in the Outer Ring Road.
The northern part of the city area (Industrial Zone, part of the Cultural Zone and the City Centre) will be drained by the main collector in the Outer Ring Road to the site next to the Irumbai Tank, which is the lowest area within Auroville. The south-west and the western part of the southern catchment area consisting of the main part of the Residential Zone, the Industrial Zone, the south-western part of the City Centre and the Matrimandir Area can also be drained to this location.
The remaining part of the southern catchment area, the eastern and the south eastern part, have to be drained in the opposite direction, to the lowest area in the east. The remaining parts of the Residential Zone and the Cultural Zone as well as the City Centre have to be drained to the location of the second treatment plant located in the east.

6.4.2 Dimensioning of the Sewer System

Wastewater and stormwater will be drained in a separate system. The hydraulic calculations of the sewers is based on the equation of Pradtl-Coolbrook:

$$\frac{1}{\sqrt{\lambda}} = -2\lg\cdot\left(\frac{2,51}{Re\sqrt{\lambda}} + \frac{k}{3,71 \cdot D}\right)$$

$$Q = A\cdot\left(-2\lg\left[\frac{2,51\nu}{D\cdot\sqrt{2gJD}} + \frac{k}{3,71\cdot D}\right]\right)\sqrt{2gJD}$$

The roughness was estimated to be:

k_b = 1,5 mm

For the dimensioning of the sewers, the tables of Dr.-Ing. P. Unger were used.
For the stormwater sewers, pipes with diameters (DN) of 200 mm to 500 mm were selected.
The minimum depth of the inverted level was chosen to be:

t_{min} = 1,7 m

The average depth of the inverted level was chosen to be:

t = 2,2 m.

The maximum depth was found to be:

4,5 m.

The depth of the household connections will be between:

1,5 m and 2,0 m.

The minimum slope is I = 1 : DN (for DN 200 at 1 : 200 and for DN 500 at 1 : 500).
For house connection pipes with a diameter of DN 150 a minimum slope of I = 1 : 50 should be used.
The sewer line should be located on one side of the road. The pipe should be made of waterproof stoneware and the manholes constructed from waterproof precast concrete elements at a distance of 50 m to 80 m.

The city can be entirely drained by gravity to the two proposed locations for the treatment plants. The required diameter of the sewers ranges from DN 200 up to DN 400. The entire length of the sewer system has been estimated according to table 6.6 at:

$$L_{tot} = 35{,}679 \text{ m}$$

Table 6.6: Length of the Sewer in Auroville

Diameter of the sewers						
DN 200	**DN 250**	**DN 300**	**DN 350**	**DN 400**	**DN 500**	
Length of the sewers						**Total Length**
[m]	[m]	[m]	[m]	[m]	[m]	[m]
22,675.56	4,168.20	5,357.00	308,00	3,170.00	0,00	**35,678.76**

The total length of the Main Collector in each of the three catchment areas can be obtained from table 6.7:

Table 6.7: Length of the Main Collector of the Sewer System in Auroville

Catchment Area	Diameter of the Main Collector					
	DN 200	DN 250	DN 300	DN 350	DN 400	
	Length of the Main Collector					**Total**
	[m]	[m]	[m]	[m]	[m]	[m]
Catchment Area North	0	387	1,854	0	1,008	**3,249**
Catchment Area SW	698	589	0	0	1,306	**2,593**
Catchment Area SE						
Main collector 1	581	0	1,035	0	0	**1,616**
Main collector 2	291	0	692	308	1,291	**2,582**
Total length	**1,570**	**976**	**3,581**	**308**	**3,604**	**10,039**

Further details can be obtained from annex 3. .....

The total length of the main collectors in the three catchment areas has been estimated at

$$L_{MC} = 10.039 \text{ m.}$$

6.5 Wastewater Treatment and Re-use

6.5.1 Location of the Treatment Plants

It is proposed to locate the effluent treatment plants (ETP) in the vacant piece of land in the GreenBelt on the northwestern boundary of the city next to the Irumbai Tank, and on the eastern boundary in that part of the GreenBelt, that has sufficient space for the setting up of the proposed ETP.

If the site next to the outer ring road is chosen, the distribution of the irrigation water can be done by gravity and the energy requirement for the pumping of the effluent into the city is minimised. In order to minimise the risks of groundwater pollution through damaged pipes and accidents at the treatment plants, it can be recommended that the treatment plants are located next to the city boundary and the city boundary bund. Since the topographical map of the GreenBelt has not been completed, the optimal location still has not been identified.

6.5.2 Description of Project Components

6.5.2.1 Overall Scheme

The entire wastewater management system for the re-use of treated wastewater for irrigation purposes consists of nine major components. Two additional components need to be included if the treated wastewater is to be re-cycled and brought back into the supply stream of process water.

The major components for the re-use of treated wastewater for irrigation are listed below. These are illustrated and detailed in figures 6.5 and 6.6 and drawings nos. 42.3, 1.2.3 and 4.

1. Pumping Stations (PS) are not required since the topography provides sufficient slope for the inflow into the treatment plant to be by gravity to the effluent treatment plant (ETP), if it is located next to the Ring Road. They may be required if the Treatment Plant is located next to the city boundary bund,

2. Screen and Grit (SG) removal system for the removal of large suspended and floating matter as well as grit and sand,

3. Imhoff Tank (IT) system for the primary treatment and cleaning of wastewater as well as digestion of primary and secondary sludge,

4. Trickling Filter (TF) for the partial secondary treatment of wastewater,

5. Dortmund Tank (DT) for the secondary cleaning of treated wastewater from Trickling Filter,

6. Root Zone Treatment System (RZTS) for the final secondary and tertiary treatment of wastewater,

7. Sludge Drying (SD) system for the final treatment of sludge,

8. Storage Tank (ST) system for the storage of treated wastewater suitable for irrigation,

9. Pumping system for the supply and distribution of treated wastewater.

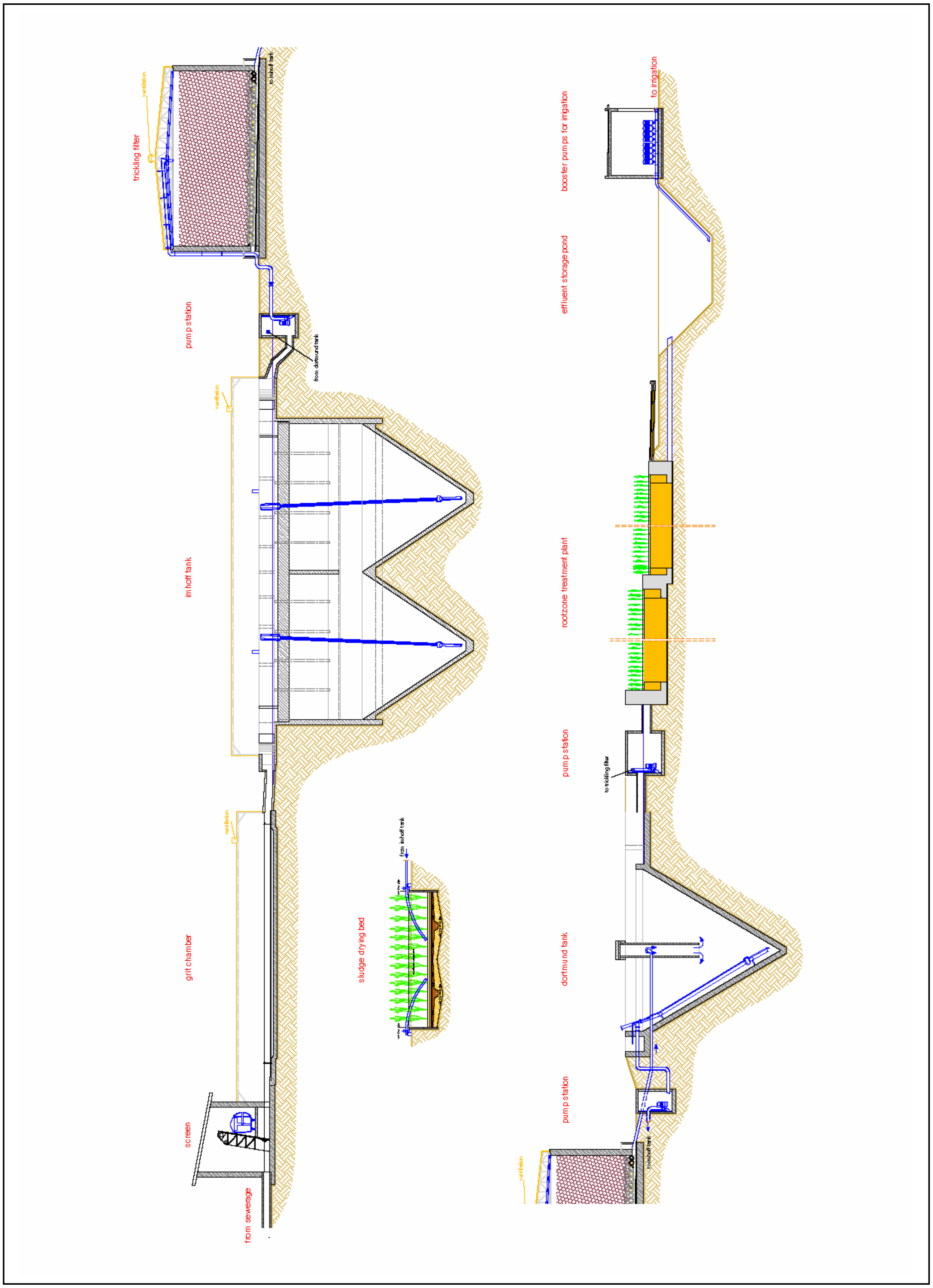
Additionally, if the treated wastewater has to be re-cycled and brought back into the supply stream of process wate,r then two additional systems have to be provided for:

10. Desalination system for the removal of excess salts from the treated wastewater,

11. Additional Storage Tank (ST) system for the storage of desalinated treated wastewater suitable for re-cycling as process water.

Figure 6.5: Scheme of Wastewater Treatment and Re-use for the City of Auroville

Figure 6.6: Scheme of the Wastewater Treatment Process



6.5.3 Location and Dimension of the Treatment Plants

The city area can be drained to two locations in the GreenBelt, one in the west and one in the east of the city.
The Wastewater Treatment Plant in the west receives the wastewater from 16,000 PE from the northern drainage areas and the wastewater from 34,000 PE from the southwestern drainage areas, making a total of 50,000 population equivalents.
The Wastewater Treatment Plant in the east receives the wastewater from a smaller catchment area of only 17,000 population equivalents (PE).

Table 6.8: Discharge of the Main Collectors of the Treatment Plants

Drainage Area	Population Equivalents
Wastewater Treatment West:	
Northern Main Collector:	
Cultural Zone	200 P
Industrial Zone	1,800 P
City Centre	1,500 P
Green Belt	1,000 P
Commercial	
Cultural Zone and City Centre	1,500 P
Industrial Zone	10,000 P
Subtotal	16,000 P

Drainage Area	Population Equivalents
South-Western Collector:	
Parts of the Residential Zone	28,000 P
City Centre	2,400 P
International Zone	600P
Green Belt	500 P
Commercial	
Residential Zone and City Centre	1,000 P
International Zone	1,500 P
Subtotal	34,000 P
Total WWTP West	50,000 P

Wastewater Treatment Plant East:

Parts of the Residential Zone	12,500 P
Cultural Zone	400 P
City Centre	600 P
Green Belt	500 P
Commercial	
Cultural Zone and City Center	3,000 P
Total WWTP East	17,000 P

The Wastewater Treatment Plant located in the West can be constructed in 3 units, one for the separate treatment of wastewater from the Industrial Zone with a capacity of 16,000 PE, the other two dealing.
mainly with domestic wastewater of in 17.000 PE each.
The total treatment capacity of the Wastewater Treatment Plant located in the West is therefore 50,000 PE, while
the Wastewater Treatment Plant located in the East can be designed as one unit with its capacity of 17,000 PE.

6.5.4 Description of the Wastewater Treatment Plant

6.5.4.1 Screen and Grit Removal System

The screen and grit removal system consists of a split channel with bar screens at each inflow of the channel. The screens remove large suspended and floating material and the channels collect the grit that settles down from the wastewater.

For the removal of suspended and floating matter of > 50 mm a bar screen system is proposed. It is estimated that from a total volume of wastewater of 963,600 and 1,023,825 m^3/a with a maximum peak flow of 188,6 m^3/h and 200,4 m^3/h, about 48 m^3/a and 51 m^3/a of screened waste material will be collected in the screen. The bar screen system will be integrated with a grit removal system. It is estimated that about 192 m^3/a of grit from the same volume of wastewater will be collected in this system. This is detailed in annex no. 3.2.1.

As wastewater from the long sewer lines would have started becoming septic it would generate odours. Thus the screen and grit removal chamber must be covered. Gases collected over this system will need to be conveyed to a biological air purification system for odour removal.

6.5.4.2 Imhoff Tank System

The Imhoff tank consists of a two-storey tank in which sedimentation is accomplished in the upper compartment, and digestion of settled solids in the lower compartment. The upper compartment contains longitudinal sedimentation chambers in which water flows horizontally. Settled solids pass through an opening in the bottom of the settling chamber and are deposited in the lower compartment for digestion. Anaerobic digestion of the settled solids is accomplished in the digestion chamber. The digestion chamber will also receive secondary sludge drawn from the Dortmund Tank system.

The digested solids are removed through a sludge drawing mechanism. Gases that are produced in the digester are collected beneath the sedimentation chamber. These gases are either dispersed or can be collected for use as biogas. The entire Imhoff tank system is covered and ventilated. The ventilated emissions from the Imhoff tank are fed into a biological air purifying system to remove odours.
For the primary treatment and sedimentation of wastewater as well as digestion of primary and secondary sludge, an Imhoff tank is proposed. This is designed to cater to a PE of 8,000 up to 9,000 for one unit, giving a wastewater flow of 963,600 and 1,023,825 m³/a for a double unit Imhoff tank with a maximum peak flow of 188,6 and 200,4 m³/h and a pollution load of 960 and 1,020 kg/d BOD_5.

The proposed Imhoff tank will have a sedimentation chamber of 80 m² with a cross-section area 25 m² and a digester volume of 375 m³ for one unit and 750 m³ for the double unit. This is detailed in annex no. 3.2.2.

6.5.4.3 Trickling Filter System

The Trickling Filter is an aerobic attached-growth treatment system. It consists of a highly permeable medium to which microorganisms are attached and through which wastewater is percolated or trickled. A wastewater distribution system evenly spreads the wastewater on top of the media and an underground drain system collects the wastewater and any biological solids that have become detached from the media. The entire Trickling Filter is covered and ventilated.

For partial secondary treatment of wastewater a single Trickling Filter is proposed. The proposed Trickling Filter is designed to cater to a PE of 16,000 and 17.000, with a wastewater flow of 963,600 and 1,023,825 m³/a with a maximum peak flow of

188,6 and 200,4 m³/h, and a pollution load of 960 and 1,020 kg/d BOD_5. The trickling filter is designed to provide partial treatment.

The proposed Trickling Filter will contain approximately 800 m³ and 850 m³ of filter material that consists of plastic medium with a specific area of 100 m²/m³. This unit will cover an area of approximately 200 m². This is detailed in annex no. 3.2.3. The trickling filter will be covered, and gases emitted from the system will be conveyed to a biological air purification system for removal of odours.

6.5.4.4 Dortmund Tank System

The Dortmund Tank is a conical secondary clarification tank used for the sedimentation of settleable solids. Part of the effluent is re-circulated to the Trickling Filter and part is directed to the RZTS for final treatment. Secondary sludge that is collected at the bottom of the tank is drawn and pumped into the digestion chamber of the Imhoff tank for digestion.

The proposed Dortmund Tank is designed to cater to a PE of 16,000 and 17,000, giving a wastewater flow of 963,600 and 1.023.825 m³/a with a maximum peak flow of 188,6 and 200,4 m³/h. This is detailed in annex no. 3.2.3.

6.5.4.5 Root Zone Treatment System

The Root Zone Treatment Plant (RZTP) is a sealed filter bed (also known as the reed bed) consisting of a sand/gravel/soil system, occasionally with a cohesive element, planted with vegetation that can grow in wetlands. After removal of coarse and floating material, the wastewater passes through the filter bed, where biodegradation of the wastewater takes place.

The functional mechanisms in the soil matrix that are responsible for the mineralization of biodegradable matter are characterized by complex physical, chemical and biological processes, which result from the combined effects of the filter bed material, wetland plants, micro-organisms and wastewater.

The treatment processes are based essentially on the activity of micro-organisms present in the soil. The smaller the grain size of the filter material, and consequently larger the internal surface of the filter bed, the higher would be the content of microorganisms. Therefore the efficiency should be higher with finer bed material. This process, however, is limited by the hydraulic properties of the filter bed. The finer the bed material the lower the hydraulic load and the higher the clogging tendency. The optimization of the filter material in terms of hydraulic load and bio-degradation intensity is therefore the most important factor in designing the RZTP.

The oxygen for microbial mineralization of organic substances is supplied through the roots of the plants, by atmospheric diffusion and in the case of intermittent wastewater by feeding through suction into the soil caused by the out-flowing wastewater. The roots of the plants intensify the process of biodegradation also by creating an environment in the rhizosphere, which enhances the efficiency of micro-organisms and reduces the tendency of clogging of the pores of the bed material caused by an increase of biomass.

RZTP contain aerobic, anoxic and anaerobic zones. This, together with the effects of the rhizosphere, causes the presence of a large number of different strains of micro-organisms, and consequently a large variety of bio-chemical pathways are formed. This explains the high efficacy of bio-degradation of substances that are difficult to treat.

The filtration by percolation through the bed material is the reason for the very efficient reduction of pathogens, depending on the size of grain of the bed material and thickness of filter, thus making the treated effluent suitable for re-use.

Conversion of nitrogen compounds (nitrification / de-nitrification) occurs due to planned flow of wastewater through anaerobic and aerobic zones.

Reduction of phosphorous depends on the availability of acceptors like iron compounds and the redox potential in the soil.

The proposed RZTP will be used for the final secondary and tertiary treatment of wastewater, particularly for the removal of pathogenic germs. **It should be noted that no other treatment system, without the use of additional chemicals or physical processes, can ensure the extensive elimination of pathogenic germs.**

The proposed RZTP consists of 50 modules for the Western WWTP and of 17 modules for the Eastern WWTP. Each module has an area of about 1000 m^2 and is divided into two stages. The first stage consists of a reed bed for high pollution and low hydraulic loads. The second stage consists of a reed bed for low pollution and high hydraulic loads. At the beginning and end of each stage and reed bed, wastewater collection and distribution chambers are located.

From past experience in tropical conditions it has been established that a horizontal filter bed area of about 1 m^2/PE (it should be noted that while the horizontal filter bed area is used as a common and convenient parameter for the dimensioning of RZTP, this is not the only parameter to be considered) is sufficient for the complete secondary and tertiary treatment of wastewater, including the removal of pathogenic germs. Therefore, the same dimensioning criteria (together with other parameters) will also be applied for the proposed RZTP.

However, it should be noted that as the proposed RZTP has never been executed in the scale and magnitude suggested in a similar location, the dimensioning parameters could be suitably revised after implementation of the initial phase(s). Nevertheless, it is expected that with increasing dimensions and magnitudes of the RZTS, the required area per PE could possibly reduce to less than 1 m^2/PE as a result of synergistic effects.

Given the criteria that the proposed RZTP will have a horizontal filter bed area of about 1 m^2/PE, the area requirement for RZTP is indicated in table 3.2.4.

6.5.4.6 Sludge Drying System

The proposed sludge drying system consists of sealed reed beds in which the digested sludge is deposited for dewatering. Digested sludge drawn from the Imhoff tank will be deposited in this system. With the help of reeds, the digested sludge will be dewatered and further mineralized. About 300 m^3 of dry sludge will have to be removed every year. This can be composted together with garden wastes and used as manure for horticultural purposes.
Annex 3.2.4 indicates the area required for the sludge drying beds.

6.5.4.7 Storage Tank System

For the storage of treated wastewater for re-use in irrigation a storage pond of 5,800 m^3 and 2.100 m^3 (WWTP West) and of 2.600 m^3 (WWTP East) is proposed. This is shown in drawing no 42.03 / 1.2.3 & 4.

6.5.4.8 Pumping System

For the supply and distribution of treated wastewater for re-use in irrigation a pumping system is proposed.
Additionally, if the treated wastewater has to be re-cycled and brought back into the supply stream of process water, then additional components have to be provided.

6.5.4.9 Additional Storage Tank System

In the case of re-cycling of treated wastewater and re-use as part of the supply stream of process water, an additional storage system will have to be provided.

6.5.5 Dimensioning of the Wastewater Treatment Plant

6.5.5.1 Inflow

a) 16,000 Population Equivalent (PE)

Q_d = 150 l/P x d x 16,000 PE
= 2,400 m³/d

Q_f = 10 % Q_d
= 240 m³/d

Qt = $Q_d + Q_f$
= 2,400 m³/d + 240 m³/d
= 2,640 m³/d

Q_{Sp14} = Q_t / 14
= 2,640 m³/d / 14 d/h
= 188,57 m³/h
= 0,052 m³/s

b) 17.000 Population Equivalent (PE)

Q_d = 150 l/P x d x 17,000 PE
= 2,550 m³/d

Q_f = 10 % Q_d
= 255 m³/d

Qt = $Qd + Q_f$
= 2,550 m³/d + 255 m³/d
= 2,805 m³/d

Q_{Sp14} = Q_t / 14
= 2,805 m³/d / 14 d/h
= 200,36 m³/h
= 0,056 m³/s

6.5.5.2 Screen Grit Removal System

Estimate of the required screen surface

a) 16,000 Population Equivalent

A_R = V_{max} / [W_h * (1-Φ / 100 %) * a]

= 0,0524 m³/s / [1,2 m/s * (1-0,6) * 0,75]

= 0,105 m²

b) 17,000 Population Equivalent

A_R = V_{max} / [W_h * (1-Φ / 100 %) * a]

= 0,0557 m³/s / [1,2 m/s * (1-0,6) * 0,75]

= 0,111 m²

6.5.5.3 Imhoff Tank System

a) 16,000 Population Equivalent

Double Tank

Q_{Sp14} = 188,6 m³/h / 2

V_S = Q_{Sp14} * t
V_S = 94,30 m³/h * 2 h
= 188,6 m³

V_{dig} = 50 l/PE * 8,000 PE
= 400 m³

b) 17,000 Population Equivalent

Q_{Sp14} = 200,4 m³/h /2

V_S = Q_{Sp14} * t
V_S = 100,2 m³/h * 2 h
= 200,4 m³

V_{dig} = 50 l/PE * 8, 500 PE
= 425 m³

V_s Volume of sedimentation chamber
V_{dig} Volume of digester

6.5.5.4 Trickling Filter System

Treatment Capacity 50%

a) 16,000 Population Equivalent

BOD_5 = 20 g/d * 16,000 PE

= 320 kg/d

B_R = 0,4 kg / m³ * d

C_m = 100 g/m³

q_A = 1,07 m/h

b) 17,000 Population Equivalent

BOD_5 =20 g/d * 17,000 PE

= 340 kg/d

B_R = 0,4 kg / m³ * d

C_m = 100 g/m³

q_A = 1,07 m/h

BOD_5 Biological oxygen demand in 5 days in g/d

B_R BOD_5-space loading (see **...0,4kg/m³d for cleaning without nitrification and plastic-filter medium)

C_o BOD_5 concentration

q_A surface flow rate (see ATV 135...0,6 up to 1,0 m/h)

V_{TK} trickling filter-volume

H calculated height of trickling filter medium

a)

$$V_{TK} = \frac{B_{dBOD5}}{B_R}$$

$$V_{TK} = \frac{320 kg/d}{0,4 kg/m^3 * d}$$

$$= 800 \text{ m}^3$$

$$H = \frac{15 * q_{A(1+RV)} * C_m}{1000 * B_R}$$

$$H = \frac{15 * 1,07 m/h * 100 g/m^3}{0,4 kg/m^3 * d}$$

$$H = 4m$$

b)

$$V_{TK} = \frac{B_{dBOD5}}{B_R}$$

$$V_{TK} = \frac{340 \ kg / d}{0,4 kg / m^3 * d}$$

$$= 850 \text{ m}^3$$

$$H = \frac{15 * q_{A(1+RV)} * C_m}{1000 * B_R}$$

$$H = \frac{15 * 1,07 m/h * 100 g/m^3}{0,4 kg/m^3 * d}$$

$$H = 4m$$

6.5.5.5 Dortmund Tank System

Side Slope 1 : n = 1,7 : 1

a) 16,000 Population Equivalent

Q_{Sp20} = 188,6 m³/h

J_{SV} = 100 ml/g

q_A = 1 m/h

F_{NK} = Q_{Sp14} / q_A
= 188,6 m³/h / 1 m/h
= 188,6 m²

VS_V = TS_R * J_{SV}
= 3 * 100 ml/l
= 300 ml/l

t_R = 2,0 h

$$V_1 = \frac{TS_R * J_{SV}}{1000} * F_{NK}$$

$$V_1 = \frac{3 \quad *100}{1000} * 188{,}6 \text{ m}^2$$

V_1 = 31,2 m³

V = Q_{Sp14}* t_R
= 377,1 m³

$$d = \sqrt{\frac{4 * F_{NK}}{\Pi}}$$

= 13,10 m

h_{total} = d/2*rake+0,5
= 13,10 m /2 *1,7+1,5
= 12,63 m

b) 17,000 Population Equivalent

Q_{Sp20} = 200,4 m³/h

J_{SV} = 100 ml/g

q_A = 1 m/h

F_{NK} = Q_{Sp14} / q_A
= 200,4 m³/h / 1 m/h
= 200,4 m²

VS_V = TS_R * J_{SV}
= 3 * 100 ml/l
= 300 ml/l

t_R = 2,0 h

$$V_1 = \frac{TS_R * J_{SV}}{1000} * F_{NK}$$

$$V_1 = \frac{3 \quad *100}{1000} * 200{,}4 \text{ m}^2$$

V_1 =33,9 m³

V =Q_{Sp14}* t_R
=400,7 m³

$$d = \sqrt{\frac{4 * F_{NK}}{\Pi}}$$

=13,50 m

h_{total} =d/2*rake+0,5
=13,50 m /2 *1,7+1,5
=12,98 m

6.5.5.6 Root Zone Treatment Plant

A_0 = 1,0 m²/PE

a) 16,000 Population Equivalent

A_{RZTP}= 16.000 m²

b) 17,000 Population Equivalent

A_{RZTP}= 17,000 m²

6.5.5.7 Sludge Drying and Composting System

Sludge Drying

Sludge generation (dry)	16,000 E / 6 PE/m² = 2.667 m²
Sludge generation (dry)	17,000 E / 6 PE/m² = 2.833 m²
Maximum stable height of sludge	h = 2,0 m
Required area for sludge drying - 5 years	A_{SVE} = 8,500 m² WWTP West
Required area for sludge drying - 5 years	A_{SVE} = 3,000 m² WWTP East

6.6 Description of the Treatment Plant West

The Wastewater Treatment Plant, located in the West of the city, can be constructed in 3 units. One unit for the separate treatment of wastewater from the Industrial Zone and parts of the Cultural Zone with a capacity of 16,000 PE.
The remaining two units are required for the treatment of domestic wastewater from the International and Residential Zone. One unit with a capacity of 17,000 PE and a second unit with a treatment capacity of 17,000 PE are required.
The total treatment capacity of the Wastewater Treatment Plant located in the West of the city is proposed to be 50,000 PE.
The wastewater flow has to be split into three streams and directed to the preliminary treatment. The first step in the treatment will be the screen. The screen will be followed by the grit channels. The screens remove large suspended and floating material. In the grit channels, the grit that can be settled will be removed from the wastewater flow.

From there the wastewater flow will be directed to the primary treatment. The grit channel will be followed by a double-unit Imhoff tank. In the Imhoff tank, the settleable solids will be settled in the upper part of the tank. One unit has a capacity of 8,000 PE up to 9.000 PE. The double-unit Imhoff tank can treat the wastewater of one treatment plant unit (16,000-17,000 PE). The Imhoff tank represents a two-storey tank in which sedimentation is accomplished in the upper compartment and digestion of settled solids in the lower compartment (3 units of w/l/h: 11 m x (2 x 8m) x 16,9 m).

The primary treatment is followed by the secondary treatment. For the biological purification a two stage process is proposed. The first stage of the treatment should be accomplished in a trickling filter for the removal of up to 50% of the pollution load (1 unit of ∅/h: 15,6 m x 4,2 m; 2 unit of ∅/h: 16,2 m x 4,2 m).
The trickling filter should be followed by a Dortmund Tank. The Dortmund Tank is a conical vertical flow secondary clarifier used for the sedimentation of settleable solids (1 unit of ∅/h: 11,5 m x 10,3 m; 2 unit of ∅/h: 12,0 m x 10,7 m). The outflow from the secondary clarifier will be further treated in the second stage in the biological treatment process in a Root Zone Treatment Plant.
The remaining pollution load and the pathogens in the partly treated wastewater still have to be removed.

The Root Zone Treatment Plant (RZTP) is a sealed filter bed (also known as the reed bed) consisting of a sand/gravel/soil system, occasionally with a cohesive element, planted with vegetation that can grow in wetlands. After removal of settleable and floating material, the wastewater passes through the filter bed where bio-degradation takes place as well as the removal of pathogens. In the final stage the Root Zone Treatment Plant requires a filter area of 5 ha.

The sludge from the secondary clarifier has to be pumped into the primary clarifier, the Imhoff tank, and the combined sludge dried in the sludge drying beds (8.500 m^2). The proposed sludge drying system consists of sealed reed beds in which the digested sludge is deposited for de-watering. Digested sludge drawn from the Imhoff tank will be deposited in this system. With the help of reeds, the digested sludge will be de-watered and further mineralized. About 300 m^3 of dry sludge will have to be removed every year. This can be composted together with garden wastes and used as manure for horticultural purposes. The sludge drying beds require a surface of 8,400 m^2.

The treated effluent will be re-used for irrigation. To balance the supply and the demand of treated wastewater, an effluent storage pond has to be provided. For the storage of treated wastewater for re-use in irrigation, a storage tank of 5,800 m^3 for domestic wastewater and 2,100 m^3 for the industrial wastewater is proposed. Only for re-use will the two effluent streams from domestic and industrial sources be combined. For the supply and distribution of treated wastewater for re-use in irrigation a pumping system is proposed. The irrigation water is supplied in a separate pipe network to the city.
Additionally, if the treated wastewater has to be re-cycled and brought back into the supply stream of process water, then additional components for desalination have to be provided.

6.7 Description of the Treatment Plant East

The Wastewater Treatment Plant located in the East can be designed as one unit with a capacity of 17,000 PE. The treatment plant receives the water from the Cultural Zone and from parts of the Residential Zone.
The size of the treatment plant is equivalent to one unit of the Treatment Plant West. It consists of the same preliminary treatment, one double unit Imhoff tank (1 unit of w/l/h: 11 m x (2 x 8m) x 16,9 m) followed by a trickling filter (1 unit of ∅/h: 16,2 m x 4,2 m) and a Dortmund tank (1 unit of ∅/h: 12,0 m x 10,7 m) as first stage of the secondary treatment. The final purification will be achieved with a Root Zone Treatment Plant of size 17,000 m². The sludge drying beds require a size of 3,000 m².

The treated effluent will be re-used for irrigation and therefore a storage tank of 2,600 m³ capacity and a pumping system to feed the treated effluent into the irrigation pipe network is required.

6.8 Estimated Costs

6.8.1 Estimated Costs for the Wastewater Management System

The estimated costs are based generally on unit prices available from India. Wherever prices were not available, unit prices from Germany have been used. The following exchange rate has been used

1 Euro = 50 RS.

The construction costs of the Auroville Sewer Network have been estimated to be

189,552,000 RS.

The construction costs of the Wastewater Treatment Plant East have been estimated to be 121,215,000 RS.

The construction costs of the Wastewater Treatment Plant West have been estimated to be 356,515,000 RS.

The total construction costs of the Wastewater Management System have been estimated to be

667,282,000 RS.

Detailed cost estimates are presented in table 6.9 and Annex 3.3.1-3.3.3

6.8.2 Estimated Costs for Operation and Maintenance of Wastewater Management System

The annual costs for operation and maintenance of the Auroville Sewer Network have been estimated to be

636,000 RS/a.

The annual costs for the operation and maintenance of the Wastewater Treatment Plant East have been estimated to be

9,091,000 RS/a.

The annual costs for the operation and maintenance of the Wastewater Treatment Plant West have been estimated to be

26,739,000 RS.

The annual costs for the operation and maintenance of the Wastewater Management system are

36,466,000 RS/a.

The detailed cost estimates are presented in table 6.10.

6.8.3 Estimated Water Price of Wastewater Management System

The costs for 1 cubic meter of wastewater has been estimated to be

13,06 RS/m³.

The detailed cost estimates are presented in table 6.11.

6.8.4 Estimated Costs for the entire Water Management Scheme

The total costs for the entire water management scheme have been estimated to be

4,793,930,000 RS.

The detailed cost estimates are presented in table 6.9.

Table 6.9 Summary Estimate of Construction Costs

No.	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	Total Costs [RS]	Total Costs * [€]
1	DRINKING WATER SUPPLY		
1.1	CONSTRUCTION OF DRINKING WATER EXTRACTION AND TREATMENT	103,111,000	2,062,000
1.2	CONSTRUCTION OF PIPE NETWORK FOR DRINKING WATER SUPPLY	91,869,000	1,837,000
1.3	CONSTRUCTION OF PIPE NETWORK FOR PROCESS AND IRRIGATION WATER SUPPLY	131,426,000	2,629,000
1	SUBTOTAL	326,406,000	6,528,000
2	STORMWATER MANAGEMENT		
2.1	CONSTRUCTION OF THE DOMESTIC CISTERN SYSTEM	2,315,270,000	46,305,000
2.2	CONSTRUCTION OF THE STORMWATER DRAINAGE SYSTEM	59.267,000	1,185,000
2.3	CONSTRUCTION OF THE STORMWATER RUNOFF SEDIMENTATION BASIN AND STORAGE TANKS IN THE GREEN BELT	849,560,000	16,991,000
2.4	CONSTRUCTION OF THE RAINWATER FILTRATION AND CONVEYANCE FROM THE GREEN BELT TO THE CITY	82,180,000	1,644,000
2.5	CONSTRUCTION OF THE RAINWATER RECIRCULATION AND FILTRATION IN PUBLIC PARKS	99,058,000	1,981,000
2.6	CONSTRUCTION OF THE MATRIMANDIR LAKE AND THE GROUNDWATER RECHARGE	394,907,000	7,898,000
2	SUBTOTAL	3,800,242,000	76,005,000
3	WASTEWATER MANAGEMENT		
3.1	CONSTRUCTION OF THE AUROVILLE SEWER NETWORK	189,552,000	3,791,000
3.2	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT EAST	121,215,000	2,424,000
3.3	CONSTRUCTION OF THE WASTEWATER TREATMENT PLANT WEST	356,515,000	7,130,000
3	SUBTOTAL	667,282,000	13,346,000
	COSTS OF THE SANITARY INFRASTRUCTURE OF AUROVILLE	4,793,930,000	95,879,000

* ... exchange rate 1€ = 50 RS

Table 6.10 Costs for Operation and Maintenance of the Wastewater Management

COSTS OF OPERATION AND MAINTENANCE OF THE WASTEWATER MANAGEMENT			
No.	**Description**	**Population Equivalent**	**Total Annual Costs of O&M**
	[-]	**[PE]**	**[RS/a]**
Total annual Costs of O & M of the Wastewater Treatment Plant (17,000 PE)			
1	Screen and Grit Chamber	17.000	52.650
2	Imhoff Tank	17.000	499.650
3	Trickling Filter	17.000	1.626.375
4	Dortmund Tank	17.000	191.400
5	Rootzone Treatment Plant	17.000	5.825.625
6	Sludge Drying Beds	17.000	662.250
7	Pipes and Equipment	17.000	233.175
Total annual Costs of O & M of the Wastewater Treatment Plant (17,000 PE)			**9.091.125**
Total annual Costs of O & M of the Wastewater Treatment Plant (50,000 PE)			
8	Screen and Grit Chamber	50.000	154.875
9	Imhoff Tank	50.000	1.469.625
10	Trickling Filter	50.000	4.783.500
11	Dortmund Tank	50.000	562.875
12	Rootzone Treatment Plant	50.000	17.134.125
13	Sludge Drying Beds	50.000	1.947.750
14	Pipes and Equipment	50.000	685.875
Total annual Costs of O & M of the Wastewater Treatment Plant (50,000 PE)			**26.738.625**
Total Costs of Sewer Network			**636.173**
TOTAL ANNUAL COSTS FOR OPERATION AND MAINTENANCE OF THE WASTEWATER MANAGEMENT			**36.465.923**

Table 6.11 Estimate of the Costs for Water Supply

	Description	Construction Costs	Costs for M&O	Drinking Water[1] Costs / m³		Process Water[2] Costs / m³		Drinking and Process Water[3] Costs / m³		Irrigation Water[4] Costs / m³		Wastewater[5] Costs / m³	
No.	**Costs for the Sanitary Infrastructure of Auroville**	[RS]	[RS/a]	[RS/m³]	[€/m³]	[RS/m³]	[€/m³]	[RS/m³]	[€/m³]	[RS/m³]	[€/m³]	[RS/m³]	[€/m³]
1	**WATER SUPPLY ***												
1.1	Drinking Water Extraction and Treatment	103.111.000	14.009.000	4,38	0,09			2,92	0,06				
1.2	Pipe Network for Drinking Water Supply	91.869.000	5.912.100	2,11	0,04			1,41	0,03				
1.2	Pipe Network for Process and Irrigation Water Supply	131.426.000	4.440.800							3,61	0,07		
1	**SUBTOTAL**	**326.406.000**	**24.361.900**	**6,49**	**0,1**	**0,0**	**0,0**	**4,33**	**0,09**	**3,61**	**0,07**	**0,00**	**0,00**
2	**STORMWATER MANAGEMENT ****												
2.1	Domestic Cistern System	2.315.270.000	57.881.750			85,2	1,70	28,40	0,57				
2.2	Stormwater Drainage System	59.267.000	4.523.893	1,39	0,03			0,93	0,02				
2.3	Stormwater Runoff Sedimentation Basins and Storage Tanks in the Greenbelt	849.560.000	33.982.400	11,58	0,23			7,72	0,15				
2.4	Rainwater Filtration and Conveyance from the Greenbelt to the City	82.180.000	6.163.500	1,90	0,04			1,27	0,03				
2.5	Rainwater Circulation and Filtration in Public Parks	99.058.000	7.429.350	2,30	0,05			1,53	0,03				
2.6	Matrimandir Lake and Groundwater Recharge	394.907.000	29.618.025	9,15	0,18			6,10	0,12				
2	**SUBTOTAL**	**3.800.242.000**	**139.598.918**	**26,33**	**0,53**	**85,2**	**1,70**	**45,95**	**0,92**	**0,00**	**0,00**	**0,00**	**0,00**
3	**WASTEWATER MANAGEMENT**												
3.1	Sewer Lines**	189.552.000	636.173									0,69	0,01
3.2	Wastewater Treatment Plant East*	121.215.000	9.091.125									3,14	0,06
3.3	Wastewater Treatment Plant West*	356.515.000	26.738.625									9,23	0,18
3	**SUBTOTAL**	**667.282.000**	**36.465.923**	**0,00**	**0,00**	**0,0**	**0,00**	**52,05**	**1,04**	**0,00**	**0,00**	**13,06**	**0,26**
	TOTAL COSTS FOR WATER SUPPLY	**4.793.930.000**	**200.426.741**	**32,82**	**0,66**	**85,2**	**1,70**	**102,33**	**2,05**	**3,61**	**0,07**	**13,06**	**0,26**

*... Time of Depreciation 50 years **... Time of Depreciation 100 years
1)... with 3.668.250,00 m³/a 2)... with 1.222.750,00 m³/a 3)... with 2.445.500,00 m³/a for Drinking Water and 1222750 m³/a for Process Water
4)... with 1.956.400,00 m³/a 5)... with 3.668.250,00 m³/a

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