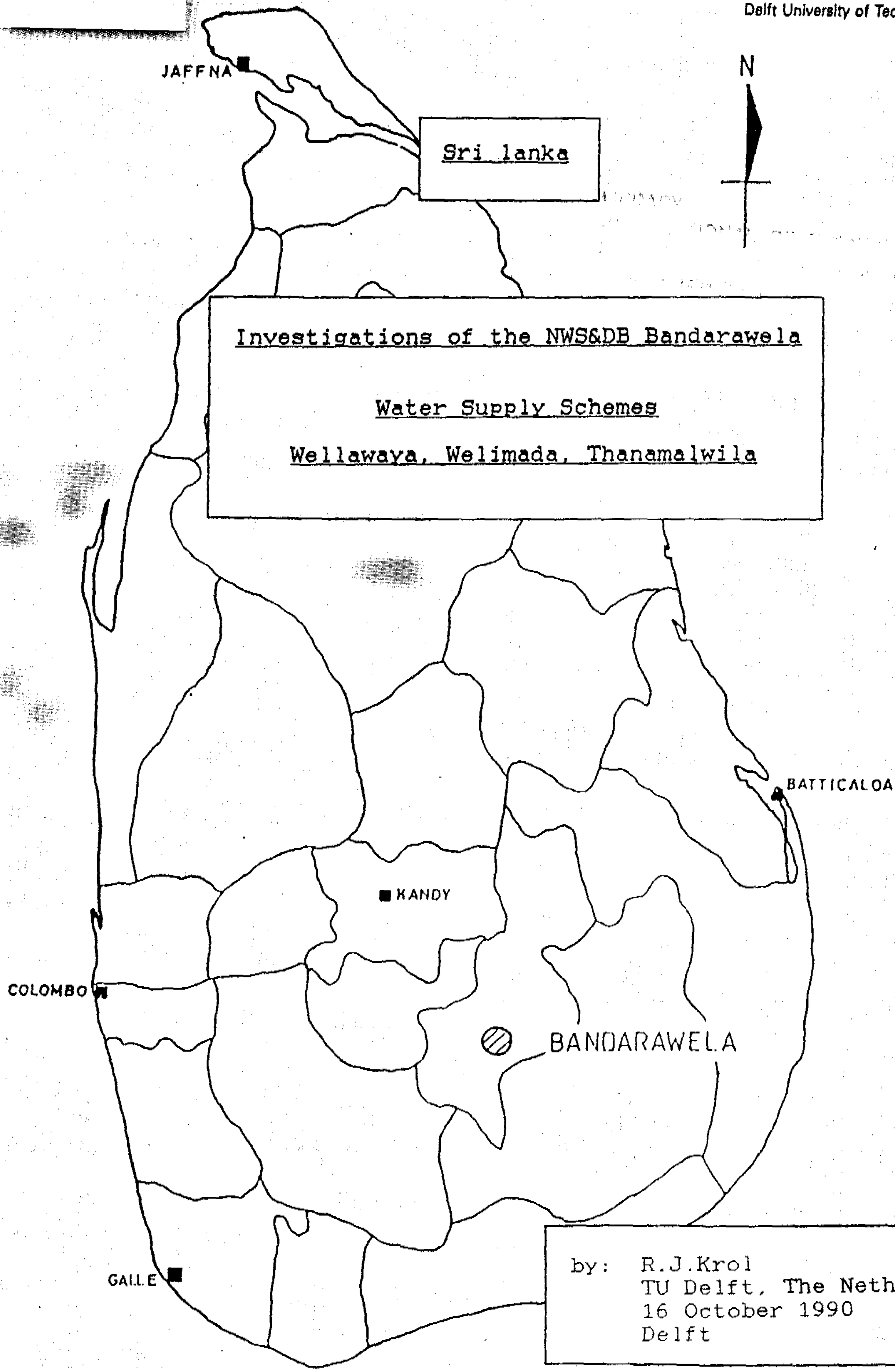
Delft University of Technology

Sri lanka

Investigations of the NWS&DB Bandarawela

Water Supply Schemes

Wellawaya, Welimada, Thanamalwila



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TU Delft, The Netherlands
16 October 1990
Delft

Preface.

This report deals with the investigations I performed, during my stay of three months at the NWS&DB in Bandarawela, Sri Lanka. Initially I went there to investigate the problem of the high turbidity contents, which occur during the rainy season, in Wellawaya and Welimada. More specifically, my purpose was to investigate the possibilities of a Horizontal Roughing Filter (HRF); this was the object I was given support for by my professor and for which I received money from my university (faculty of civil enginering, TU Delft, Holland) and from the Royal Institute of Engineers (Kivi).

At the time of my arrival, the rainy season had not started yet. At the same time, Mr. S.B.Palipana, my counterpart in Bandarawela, had some severe clogging problems with the iron content of his Thanamalwila plant. Hence, I decided to give my first attention to the problem of the iron content in the drinkingwater system in Thanamalwila. During six weeks I investigated the scheme together with Mr. H.A.K.Amarakoon, the results of which constitute part of this report.

Most of the remaining time, I spent on the investigation of the turbidity problem in Wellawaya. After some rough assumptions and estimations, I made a design for a pilot plant of an HRF. With the aid of the NWS&DB Bandarawela, I was able to start the construction work, but due to lack of time, the work was not completed when I left. When I returned to the Netherlands, I provided for an HRF pilot plant manual with the aid of my professor.

Again with the aid of my professor, I provided an URF pilot plant manual for the construction of such a filter in Thanamalwila.

I would like to thank Mr. S.B.Palipana, Mr. R.Wyesooriya and Mr H.A.K.Amarakoon, for their cooperation and capability to give feedback. Furthermore I would like to thank professor ir. J.H.Kop, the TU Delft and the Kivi, who made it possible for me to gain a lot of experience in the field of drinkingwater engineering in a foreign country. Mr.H.Heynen of the IRC I would like to thank because of his practical advices and support before and during my stay in Sri Lanka.
Furthermore I would like to thank ir. J.H.Rakers for his over-all coordination and Mr. H.Haas and Mr. D.D.C.Gamalatke for their hospitality during the first part of my stay in Sri Lanka.
Special thanks to Mr. Palipana because of the hospitality he showed me during the five weeks I stayed with his family.

R.J.Krol.

Summary.

NWS&DB Bandarawela:

After discussions with the regional manager and his assistant, there appeared to be a number of difficulties, which mainly consisted of problems with:
1) lack of standard operation procedures; caretakers did not work properly due to lack of education or motivation;
2) lack of possibilities to monitor the expenditures, consumer demands and revenues;
3) turbidity problems with the intake water, due to lack of pre-treatment devices.

Wellawaya scheme:

Two main problems appeared to exist:
1) rapid clogging of the filter, due to high turbidity of the river water during the rainy season; because of this, the water had to be bypassed during this period;
2) the intake construction was not strong enough to resist the attack of trees, that drift along the river during the rainy season.

Recommendations:
1) From the different possible alternative solutions, an HRF is recommended as a pre-treatment device. An HRF pilot plant should be built and monitored for at least one rainy season. A manual has been composed to guide construction and operation.
2) It is recommended to make a plain sedimentation basin, located in the old irrigation canal, in order to reduce the turbidity down to a level, so that the influent can be filtered by the HRF.

Welimada scheme:

Two main, possibly interconnected, problems appeared to exist:
1) The efficiency of the Ranney Well was below design level.
2) The nearby river had moved its course over the past two to three years, for about five meters. The laterals did not reach under the river bed anymore.

Recommendations:
1) Fix the river bed course by installing "gabions".
2) A lateral should be excavated to inspect the gravel layers.
3) A trench should be excavated along the river bank, in which a lateral pipe and gravel layers should be installed, in order to simplify inspection and maintenance.

Thanamalwila scheme:

Stage I:
The main problem was the high iron content of the groundwater, which caused severe clogging problems. The system to remove the iron particles did not function because of:
1) lack of maintenance of the aerator;
2) non functioning of the settlement tank;
3) addition of chlorine before distribution.
Improvement of the efficiency of the settlement tank would not be sufficiently effective because of the low settlement velocity of the particles.

Recommendations:
1) An Upflow Roughing Filter (URF) pilot plant should be installed. An URF pilot plant manual has been composed, containing guidelines for construction, operation and maintenance, monitoring and supervising procedures.

Stage II:
Recommendations:
1) Evaluate the functioning of the URF,
2) When the functioning of the URF is not satisfactory, investigate the possibility of a different water source; the nearby river,
3) The proposed 90,000 l water tower should be built,
4) Detailed design of the extension of the system should be performed.

Content

1. Introduction to the NWS&DB Bandarawela.

In april 1990, the office of the NWS&DB was in a state of transition because of a new office, which would be ready in June/July. Mr. S.B.Palipana Bsc, Regional Manager in Bandarawela, and his Assistant Manager Mr. R. Wijesooriya Bsc, gave a clear picture of theie office, in which 40 people were working.
The main technical equipment for running the office, consisted of:
- 2 pickups and 1 jeep (for site visits and payment of wages), 1 lorry and 1 tractor,
- 1 IBM compatible computer with accessories,
- 1 fotocopy-machine,
- a number of typewriters.

1.1 General outline.

A) Office, 3 departments:
1) Technical: site-officers (water-suply), mechanical foreman. A total of 12 personnel, plus a down-road section in Badulla, reporting to head-office in Kandy. Problems: communication and lack of tools.
2) Administrative: typists and clerks (general, 3). Total of 4 to 5 personnel.
3) Accounting and Commercial: A total of 9 personnel. Job: General ledger payments.

B) Laboratory, Mr H.A.K.Amarahoon showed the equipment present.
Measuring possibilities:
1) Chemical: PH, EC, Total Dissolved Solids (TDS), total) Hardness, Ca Hardness, Mg Hardness, total alcality, SO4, PO4, NO3, NO2, NH4, Cl-, Cl2, Si, Fe, F2, Al (no Mn!).
In all of Sri Lanka there are about 300 tube wells. After investigation, only 8 could sufficiently reach the Sri Lankan standards (are almost the same as the WHO standards). Most wells were contaminated with Fe, some with F2, Ca, Mg and (total) Hardness,
2) Physical: odour, temperature, turbidity and colour,
3) Biological: algaes, fungus and micro-organisms (bacteria, virusses).

The laboratory mainly analyses the biological (bacteria) parameters. At the schemes (this laboratory investigates over 50 schemes, with 12 major schemes), there are "continuous" turbidity- and Cl-measurements taken.

C) Organisation of the NWS&DB, from national to regional level:

Chairman
(Board of directors, political appointed, Colombo)

General Manager
(G.M.Colombo)

Additional General Manager
(A.G.M.Colombo, 2 members; Oper, Coop.Plann.)

Deputy General Manager
(D.G.M.Colombo, 4 members, Op.+Main, Constr, Plann.+Des, Adm.)

Assistent General Manager
(A.G.M, 12 members, Groundwater(Col), Train.+Res.(Col), Supplies (Col), Adm.(Col), Personnel (Col), Plann.(Col), Design (superior of S.Wyegoonewardene Bsc, another counterpart in Col), Sewerage Oper.(Col), Greater Colombo-area(Col), Regions Support South (Matare), Regions Support Kandy (Kandy), Regions Support North-East (Batticola))

Regional Manager Bandarawela
(R.M.Bandarawela, S.B.Palipana Bsc, Constr.+Op.+Main.)

Ass.Man.	Eng.+Constr.	Account. (supervision)	Chemist	Chiefclerk (secr.)

In total, 300 people work in the region.
In this region 38 sites are operated and maintained, plus some provisional schemes, which are under operation.

1.2 Items of concern.

- Quality guarantee is not possible, because of a lack of continuous water supply and not up to date latrine standards.
  Hygiene-education: Lack of "knowledge-maintenance" is a problem,
- Chlorinators (bleaching powder) are not properly operated and maintained,
- Due to lack of maintenance, the pipe systems are leaking, resulting in the waste of water and the chance by exterior pollution and contamination,
- Management and operation of the slow sand filter (installation of valves etc.). This might be partly solved by preparing proper standard procedures,
- The direct number of consumers is known (standposts estimated at 20-25 people per tap, and private taps) , but the exact figure of all water users (Non-Water Board schemes) is not known,
- During the rainy seasons there are turbidity problems (> 100 NTU) everywhere. The water itself is never a problem; just the treatment of it. If the regional or national engineers would design some pretreatment devices, these problems could be solved.

2. Design guidelines

In this chapter, general design guidelines will be stated. They are general in the sense that they should be taken into account when a waterscheme has to be designed, from the beginning to the end. A division has been made between standards (what is absolutely necessary and obliged) and recommendations (what should be considered). The actual report about the several sites only has to deal with the technical restrictions.

2.1 Standards.

A) Waterquality:

1) **Physical.**
Turbidity of the in- and effluent water of a slow sand filter:
Influent: <= 10 NTU
Effluent: <= 1 NTU

2) **Chemical.**
(Sri Lankan standards)

	Desirable level (mg/l)	Permissible level (mg/l)
a) Iron (total).	0.3	1.0
b) Fluoride.	0.6	1.5
c) Residual chlorine.	0.1	1.0

2.2 Recommendations.

B) Waterquantity:

1) **Population growth.**
It is important to know, to what extend the population will grow during the lifetime of the designed system. For country areas, villages, an annual growth rate of 2% is estimated, for town areas the estimated annual rate is 3%. The design population is estimated by:

$$Pd = Pp\ (1 + 0.01\ a)^Y$$

where:
Pd = design population
Pp = present population
a = annual growth rate (%)
Y = design period (years)

2) **Population, served by NWS&DB.**
This should be thoroughly investigated. Clear decisions should be taken about future policies.

3) **Water demand per capita.**

The design should be based on local data about the amount of water to be supplied per person per day. These data largely depend on socio-economic conditions and preferences of the community, such as:
- the time people can afford to spend to collect water,
- the accessibility of the water distribution points,
- whether water is used for watering livestock or agriculture.

3) **Supplying hours:**
These largely depend on the situation; ideal would be a supply of 24 hours per day, but in certain areas, this is not feasible and for the moment not necessary (no special industry requirements for example).

3) **Minimum head:**
For sufficient water supply, a minimum head of 10 m is required at every point of the system.

C) **Operation and Maintenance procedures:**

If guidelines should be of any use, it is a necessity to **educate** and **motivate** caretakers at the site. They are the ones immediately responsible for the proper functioning of the system. If they do not feel responsibile for whatever reason (ignorance, lack of payment/promotion possibilities), the system will not function.

D) **Costs:**

1) **NWS&DB Bandarawela.**
A very small budget is available at the regional level for minor rehabilitations.

2) **Funds.**
For large rehabilitation programs, such as new schemes or large adjustments of old schemes (Horizontal Roughing Filter, Upflow Filter), funding from outside will be necessary (government or various institutes).

E) **Time:**

Scheduling because of the dry seasons and for non-technical reasons, is important in order to have enough time to get into the details of a design.

3. Wellawaya

3.1 Introduction

Wellawaya is a small village of less than 6000 people (designed population in the year 2000). It is situated at the foot of the hills, about 8 km south of Bandarawela, Uva Province.

The Wellawaya water supply scheme consists out of a river intake, situated about 2 km away from two slow sandfilters, which are operated by gravity. Furthermore, a 10.000 gal. reservoir and a chlorinator house are located near the filters. Details about the site can be found in Appendix I.A.

3.2 Site Investigation

During the last monsoon the intake has been wiped away by trees drifting down the river. At the moment the intake consists of 6" PVC pipes, which are layed at the side of the stream: see fig 3.1.

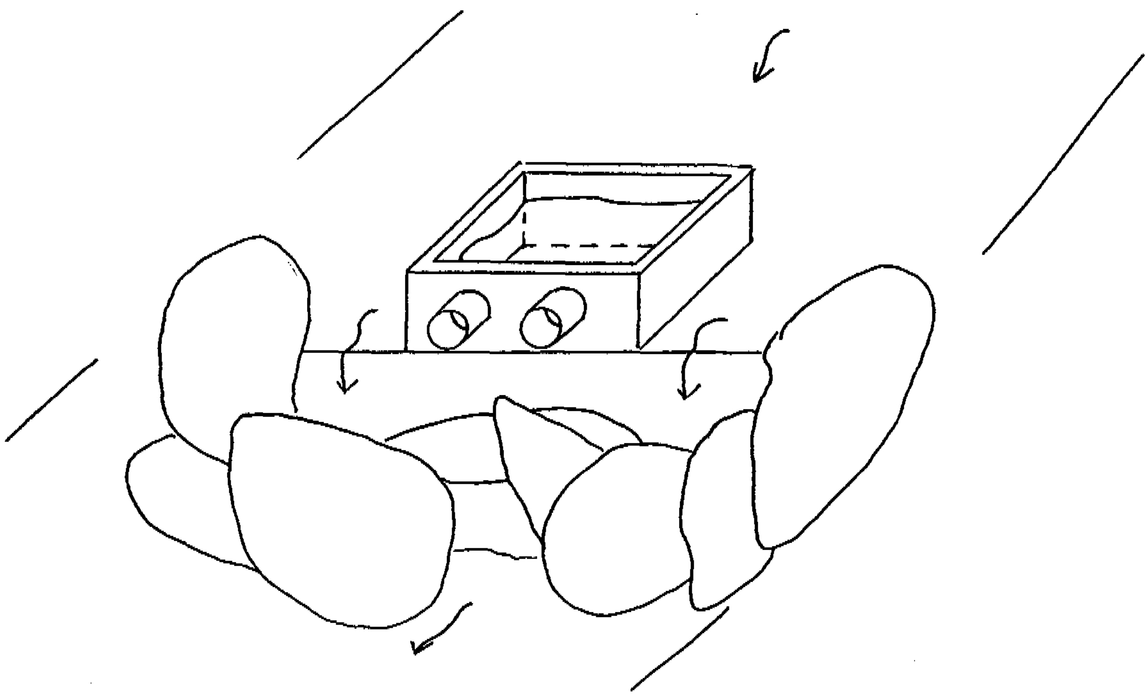


fig 3.1 Intake construction.

The intake can only be reached by foot (no vehicle or animal can supply intake materials; all material must be brought by foot). The former intake consisted of a concrete sluice in the middle of the river. Two 6" pipes (steel) transported the water to the side of the river. Nearby the river an old irrigation canal is situated. Part of it serves as a trench to place the pipes in.

The two slow sand filters are operated throughout the year. During the rainy season (heavy rains from december to february) the turbidity of the raw water rises up to such an extend, that the water has to be bypassed, in order not to damage the filters: These would clog within a day. Therefore, the water is distributed without treatment. Because of this, the consumers have to filter and boil the water themselves.

Because the site office burnt down last december, all information about the raw water characteristics were lost. However, the information of the site officers seems to indicate that a large part of the particles, which cause the high turbidity during the rainy season, consists of fine silt.

3.3 Main Problems

In this investigation only the technical side of the problems have been looked upon. Therefore two problems can be found:

1) The provisional intake: its location may be a reason for the turbidity problem.

2) The actual rise of the turbidity during the rainy season, which results in an influent turbidity for the slow sand filters, far above the admissable level (exact figures unknown).

By the design of a new intake system, these problems can be solved.

3.4 General Available Technical Solutions

A lot of experience has been gained throughout the world with the systems that are described below and that seemed most feasible for the described problem. What is said about these pre-treatment methods is just a short outline of the important principle, in order to make the right choice. For exact details, see the IRC book, to which is referred to in Chapter 6: References, nr. 4.

3.4.A Plain Sedimentation.

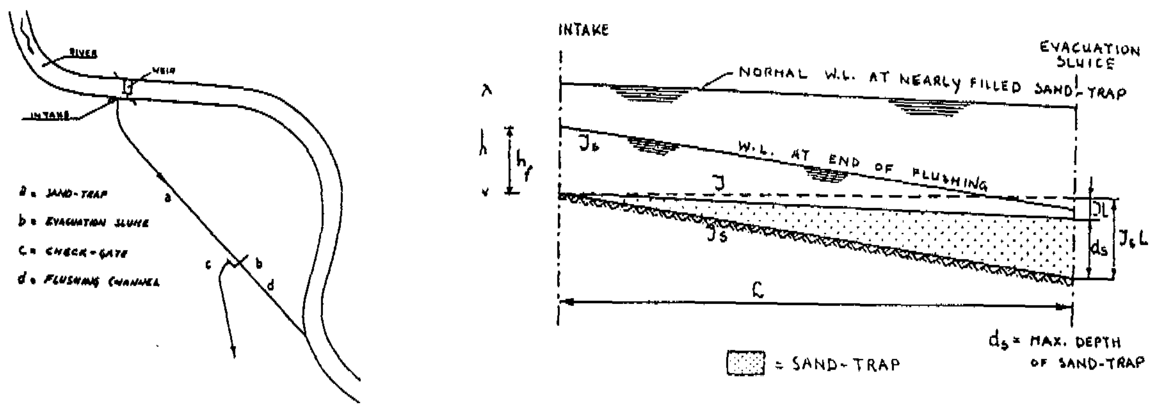


fig 3.4.1 An example of a plain sedimentation basin.

Sedimentation is the most simple way to reduce the amount of transported particles in the water. The water flows laminar in a horizontal direction through a basin. Because of the force of gravity, the particles will settle down. The efficiency of such a basin depends on the:

a) flow velocity,
b) depth and length of the basin,
c) particle size and material.

Advantages:

a) simple construction,
b) simple operation and maintenance: When the "trap" is full, it needs to be cleaned. In some cases it is possible to clean it by a sluice system, which flushes the particles out of the basin.

Disadvantages:

a) The area needed may be very large. Especially when a large part of the particles consists of silt, which can stay in suspension even in a very long basin.

3.4.B Tilted Plate Settling.

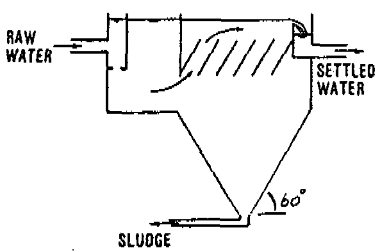


fig 3.4.2 A tilted plate settler.

This principle is a variation of the plain sedimentation system. In the basin, which is constructed in the same way as the plain sedimentation basin, several plates have been installed with some angle of inclination. By doing this, the distance the particle has to travel, is shortened to settle. This increases the efficiency. The particles will settle down at the plates and fall to the bottom when they (together) have gained some mass.

Advantages:

a) Less area required than with the plain sedimentation,
b) Simple construction,
c) Simple operation and maintenance.

Disadvantages:

a) Particles of a certain small size will not settle down because the priciple is based on the law of gravity.

3.4.C River Bed Filtration.

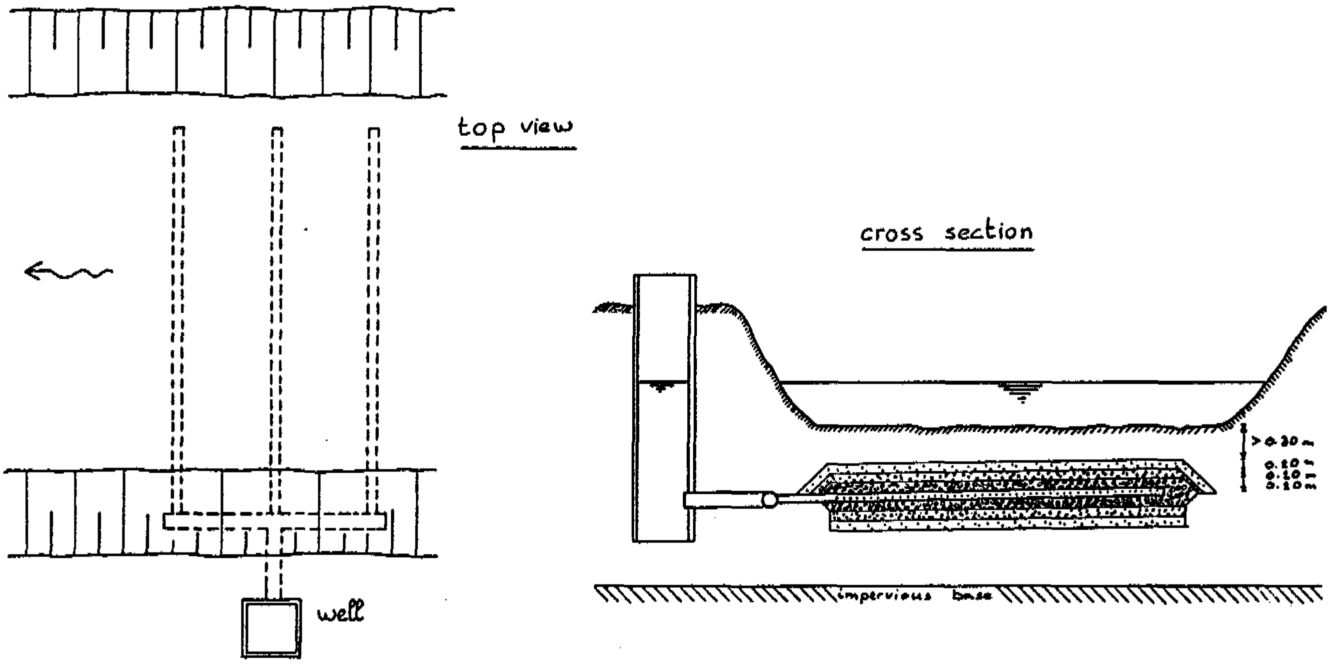


fig 3.4.3 An example of river bed filtration.

The above drawing is just one of the possibilities. The main principle is the installation of a drain beneath the river bed, which is protected by several layers of filter material. To maintain a certain rate of filtration, a minimum depth of the river water is required. Therefore several types of weirs and other constructions can be designed to raise the water level during periods of low river discharge.

Advantages:

a) Turbidity of the raw water is significantly decreased because of the straining in the filter,
b) Biological pollution of the raw water is decreased because of the bad living conditions for the micro-organisms in the filter,
c) Construction is simple but accurate,

Disadvantages:

a) When the filter clogs after a certain period (depending on the raw water characteristics and the filter material), all the filter material and the drain have to be excavated,
b) A pilot plant should be installed and monitored for at least one rainy season, to decide about the filter material and drains,
c) Because of the minimum required water depth, the river needs a sand bed in which the drains can be excavated, or else special provisions have to be taken to install the filter material.

3.4.D Modular Abstraction systems.

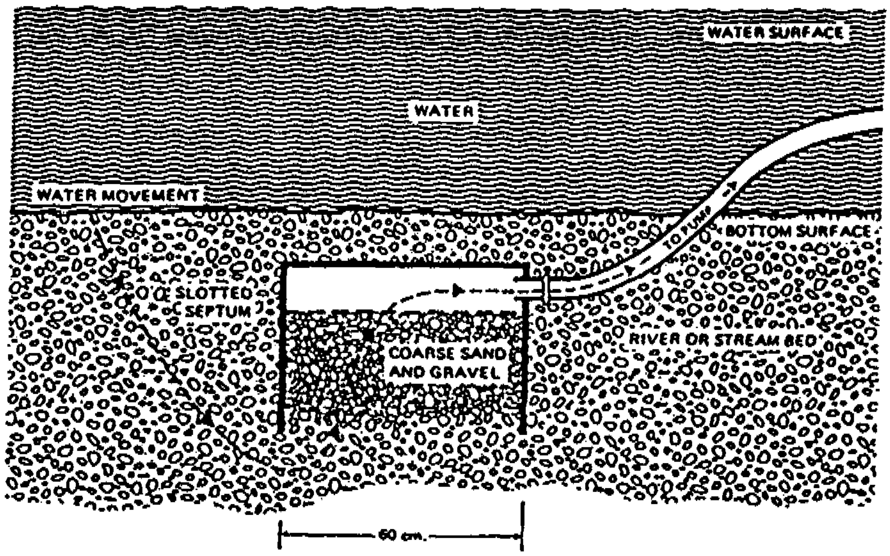


fig 3.4.4 A Modular abstraction system.

This system is a variation on the river bed filtration principle, because with the modular abstraction, the location of the filter is not restricted to the situation at the river bed anymore. The main principle is to install a box beneath, upon or even next to the river bed, through which the water has to flow in a vertical direction in order to be drained to the distribution line.
This filter is often used for small villages and camp units.

Advantages:

a) Physical and bacteriological purification,
b) Simple construction,
c) Simple operation.

Disadvantages:

a) When the filter is installed upon or beneath the river bed, it is difficult to clean the filter when it is clogged,
b) A pilot plant is needed to investigate the efficiency.

3.4.E Horizontal-flow Roughing Filter

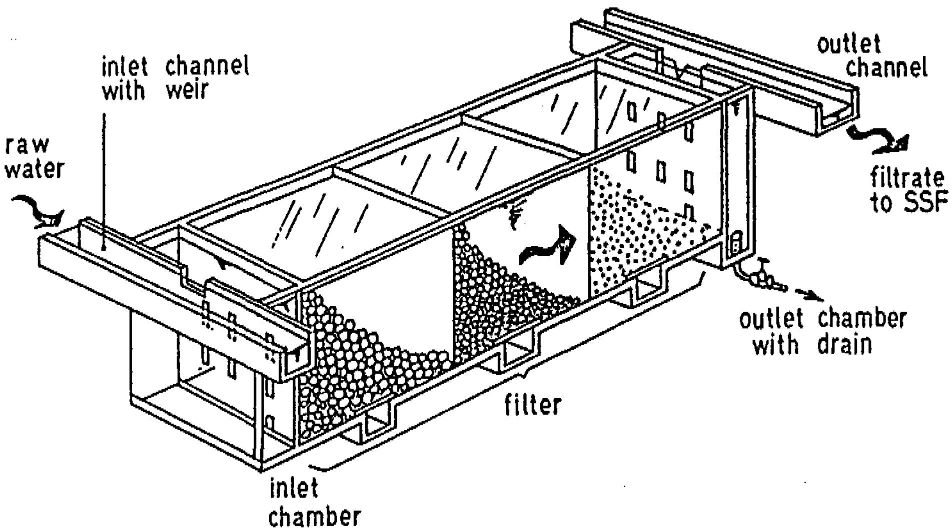


fig 3.4.5 A Horizontal Roughing Filter.

This principle is a simplification of the purification system, as it takes place in nature itself in the aquifers through which the groundwater flows.

Water flows through a horizontal basin with several compartments, which contain filter material that decreases in size in the direction of flow. The system is mainly based upon the principles of settling and straining. The particles to be removed can accumulate at the top of filter particles until a certain critical mass has been reached. When this point has been reached, the particles fall down to the bottom where they accumulate again. Because of this, the upper layers have some space left where the filter process can start again.

Advantages:

a) High filter efficiencies can be achieved,
b) The filter can be cleaned hydraulically by opening some valves at the bottom, through which the water can be flushed out,
c) When the filter is clogged after a certain period (no hydraulic cleaning is possible anymore), the filter material can be excavated and washed out more easily than when the filter was constructed in the water.
d) The filter may be operated intermittently.

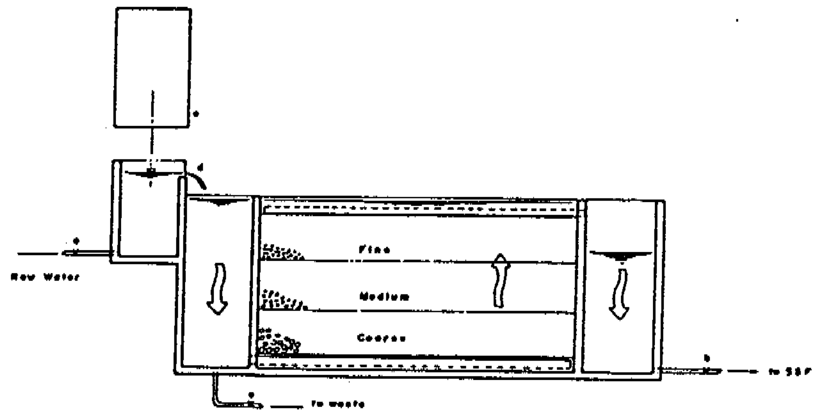
Disadvantages:

a) A caretaker should be trained to operate the filter,
b) Depending on the local situation and available materials, a basin has to be constructed,
c) A pilot plant should be installed and monitored for at least one rainy season, to decide about the filter material.

3.4.F Upflow Roughing Filter.

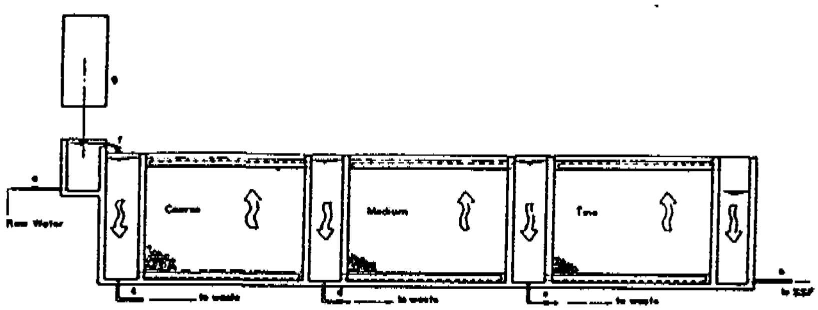
This principle is a variation to the horizontal-flow roughing filter (based upon the principles of sedimentation and strain), except for the direction of flow. Two concepts are common:

1) The water enters the filter at the bottom and flows upward untill it overflows into another compartment and after this one in another one again.
2) The water enters the filter at the bottom and flows upwards through several layers until it overflows into the outlet compartment.



a. valve for raw water inlet and regulation of filtration rate
b. effluent outlet valve
c. valve for drainage of gavel bed
d. inlet weir
e. calibrated flow indicator

fig 3.4.6 Upflow Roughing Filter, concept 1.



a. valve for raw water inlet and regulation of filtration rate
b. effluent outlet valve
c. valve for drainage of first gravel bed compartment
d. valve for drainage of second gravel bed compartment
e. valve for drainage of third gravel bed compartment
f. inlet weir
g. calibrated flow indicator

fig 3.4.7 Upflow Roughing Filter, concept 2.

With both concepts, the larger particles accumulate at the bottom. Hydraulic cleaning takes place by simply emptying the filter; a procedure which may have to be repeated a few times in a row.

Advantages:
- a) High filter efficiencies can be achieved.
- b) The filter can be cleaned hydraulically by opening some valves at the bottom, through which the water can be flushed.
- c) The filter material can be washed completely to use it again, after the filter is clogged.
- d) The filter may be operated intermittently.

Disadvantages:
- a) A caretaker should be trained.
- b) The box to be constructed should be solid, of a certain height and therefore be made with reinforced concrete, which is expensive.
- c) A pilot plant should be installed and monitored for at least one rainy season, to decide about the filter material.

3.5 Conclusions

1) Because of the fine silt particles, plain sedimentation basins or tilted plate settlers are not efficient enough to reduce the turbidity down to a level of less than 10 NTU. Such a basin may be good to retain the larger particles. The remaining silt particles should be removed by another filter construction.

2) Because of the location of the present intake, the possibilities of river bed filtration and modular sub-sand abstraction seem not to be the first choice: The intake area is difficult to reach and the river bed consists of large rocks, which exclude the river bed filtration. Therefore a filter near the slow sand filter is more practical and feasible.

3) Because of the size of the scheme (150,000 gls/day), horizontal or upflow roughing filters seem more feasible than the modular sub-sand abstraction.

4) Because of the costs and the possibility to use some surface of the nearby area (after a thorough investigation of at least one rainy season), a horizontal-flow roughing filter seems a feasible option.

3.6 Recommendations

1) A plain sedimentation basin should be installed at the intake location. The existing old irrigation canal may be used as a basin. An practical handout to design such a basin has been provided by professor ir. J.H.Kop and is available at the NWS&DB Bandarawela.

2) An HRF pilot plant should be installed at the Wellawaya site. A HRF pilot plant manual (construction, operation and maintenance procedures, monitoring and supervising programme), is presented in Appendix I.B. Details about the assumptions and estimations are presented in Appendix I.C.

4. Welimada

4.1 Introduction

Welimada is a village of about 1700 people, situated about 12 km from Bandarawela, Uva Province. The raw water intake consists of a Ranney Well with three laterals beneath the river bed, see fig 4.1. For technical details see Appendix II. When the water is collected in a sump, it is pumped uphill, whereafter it is distributed by gravity through the pipe system.

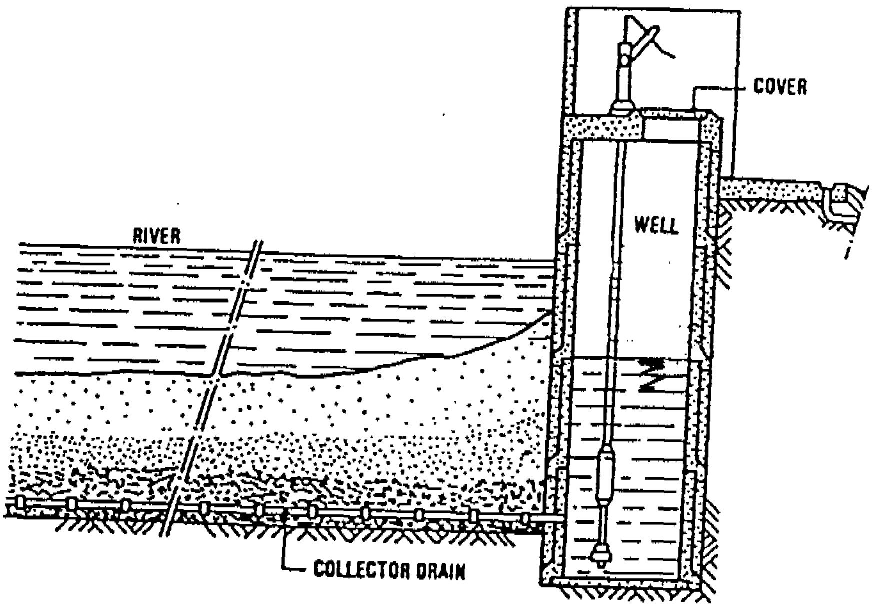


fig 4.1 A Ranney Well.

4.2 Site Investigation

Due to lack of time, only a minor investigation could be made. The following information has been gathered:

1) The 3 laterals do not reach under the river anymore; they stop at about 1 m before the riverbank. This is due to the erosion/sedimentation process, which moved the river 5 m in 2-3 years. Perhaps this was unavoidable as the intake was made on the outside of a riverbend, just at the start of the riverbend.

2) Clogging problems: for one reason or another, it was not possible to use steel laterals (price?), therefore PVC-pipes were used. Because of the strength of this material, it was not possible to drill the pipes into the ground, under the river bed, by machines. Therefore the pipes had to be digged into the river bed by hand. It appeared not to be possible to excavate deeper than 3 ft (the pipes had to be umlayered with gravel, thickness and gradation unknown).

At the moment the laterals clog easily (in fact they are clogged, but are still functioning, although far below design-level). Backwashing has been tried by means of high-pressure airpumps. Result: after 1.5 hours pumping, there was still no effect.
To get a clear picture of this problem, a lateral has to be excavated to have a look at the filter layers and the clogging of the orifices.

4.3 Main Problems

1) Efficiency of the Ranney Well is below the design level.

2) In the last 2-3 years, the river bed has moved, for about 5 m. Because of this, the laterals do not reach under the river bed any more.

4.4 Conclusions

1) Something should be done about the movement of the river bed: this should be stopped. What the relation is between the movement of the river bed and the low performance of the Ranney Well is not clear.

2) A well known problem of this kind of intake is the clogging of the laterals. This has to do with the spacing of the orifices and the covering of the laterals with gravel. For details about the design of the laterals, see reference nr. 4, chapter 6 and recommendation 2.

4.5 Recommendations

1) The erosion and sedimentation process can be stopped by fastening the direction of the flow by so called "gabions". These are rolls, mattresses, boxes etc, made of woven steel wires like telephone wires. These woven nets are filled with rocks of a size, more large as the width between the wires, see fig 4.2.

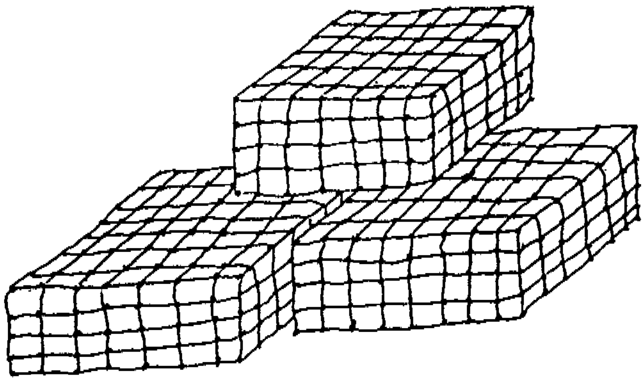
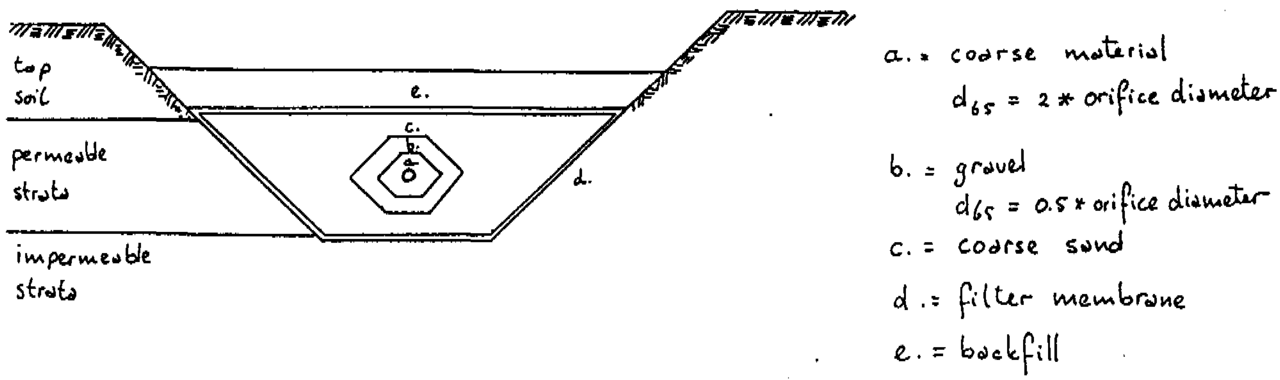
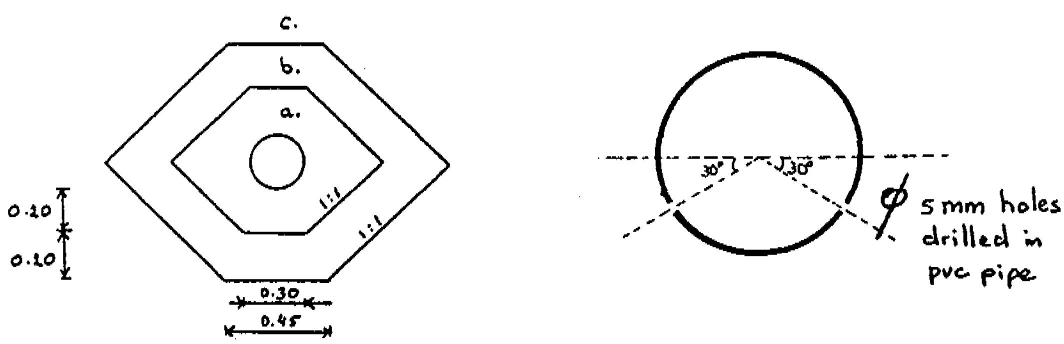


fig 4.2 An example of Gabions.

2) Dig out a lateral and investigate the gravel layer and orifices. These should be prepared like fig 4.3 shows.



Cross-section



Detail of filter

fig 4.3 Detail design of the drains.

The gravel pack composition of the filter bed depends on the raw water turbidity and the effluent requirements. Pilot testing is required. Gravel for a vertical river bed filter to treat water of a turbidity level of 500 NTU could

be as follows:
- upper layer (0.20 m), gravel size 2 - 5 mm,
- second layer (0.20 m), gravel size 5 - 10 mm,
- third layer (0.20 m), gravel size 10 - 15 mm,
- bottom layer (0.20 m), gravel size 15 - 25 mm.

The gravel in the bottom layer should be two to three times larger in diameter than the orifices.

The diameter of the main and lateral collectors should be calculated for a velocity between 0.1 and 0.5 m/s. The drains are laid at a slope of 1% towards the main and at intervals of 1.0 to 1.5 m. Orifices of 5-10 mm diameter are drilled in two lines at a 30^0 angle with the horizontal line at 150 mm intervals. (out of Ref.nr. 4, chapter 6).

3) Investigate the level of the groundwater. When this is not high, there may be not enough head for the water to press its way through the orifices. This depends on the permeability of the riverbed. When there appears to exist insufficient head, the laterals should be dugged out and placed at a deeper level. Further study should be made about the necessary head and depth of the laterals.

4) To investigate a simple alternative, excavate a trench along the river side (in the upstream direction), 1 m out of the river side, 1 m wide, see fig 4.4.

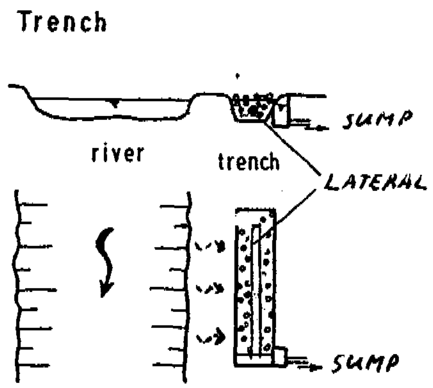


fig 4.4 Trench along the riverbank.

Fill this trench with a lateral and cover it with gravel as in fig 4.3.

The main advantage of this system is the simplicity of the excavating of the lateral, when the efficiencey is reduced due to clogging. This lateral should be investigated for a certain period to gather information about the performance.

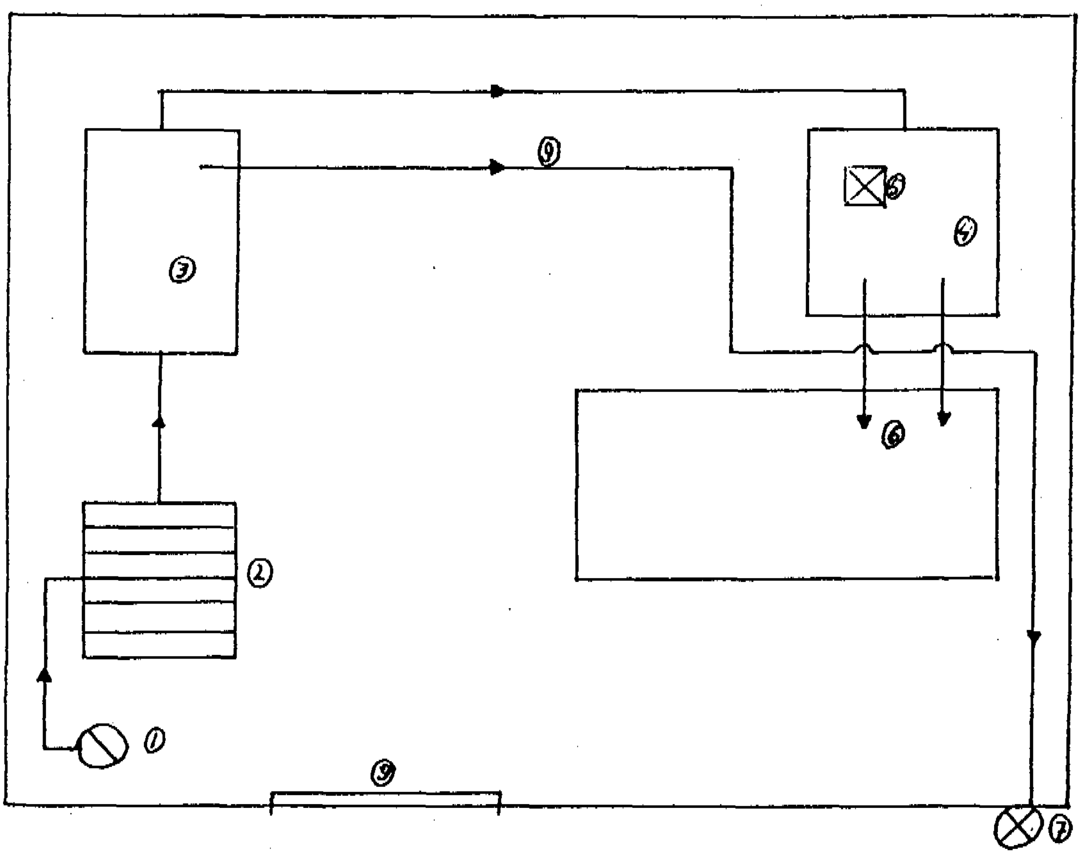
5. Thanamalwila

5.1 Introduction.

Thanamalwila is a small town on the Wellawaya-Tissa road, at the southern fringe of Moneragala District, Uva Region. According to a survey in the midst of 1980, the existing population of the proposed water served area was about 1600, including 600 residents of Sarvodaya Camp, located in the vicinity of the town.

During Stage I, the present situation, the site has been designed in attribution to the existing system of the hospital with her 15000 gls. watertower. It's location is situated just outside the town (fig. 5.1). At Stage II, the system has to be extended, c.q. the building of a 90000 l water tower, design of the extension lines and the evaluation of the sustainability of the watersource.

Tanamalwila gets its water from an aquifer. The overall system consists of a well, aeration tank, sedimentation tank, reservoir and chlorinator (fig. 5.2).



1. Well + Lowlift Pump
2. Aerator
3. Settlement Tank
4. 5000 gls. Reservoir
5. Chlorinator
6. Pumphouse + 2 Highlift Pumps
7. Standpost-tap
8. Non-designed Supply Line
9. Gate

fig. 5.2 Thanamalwila w.s.s.

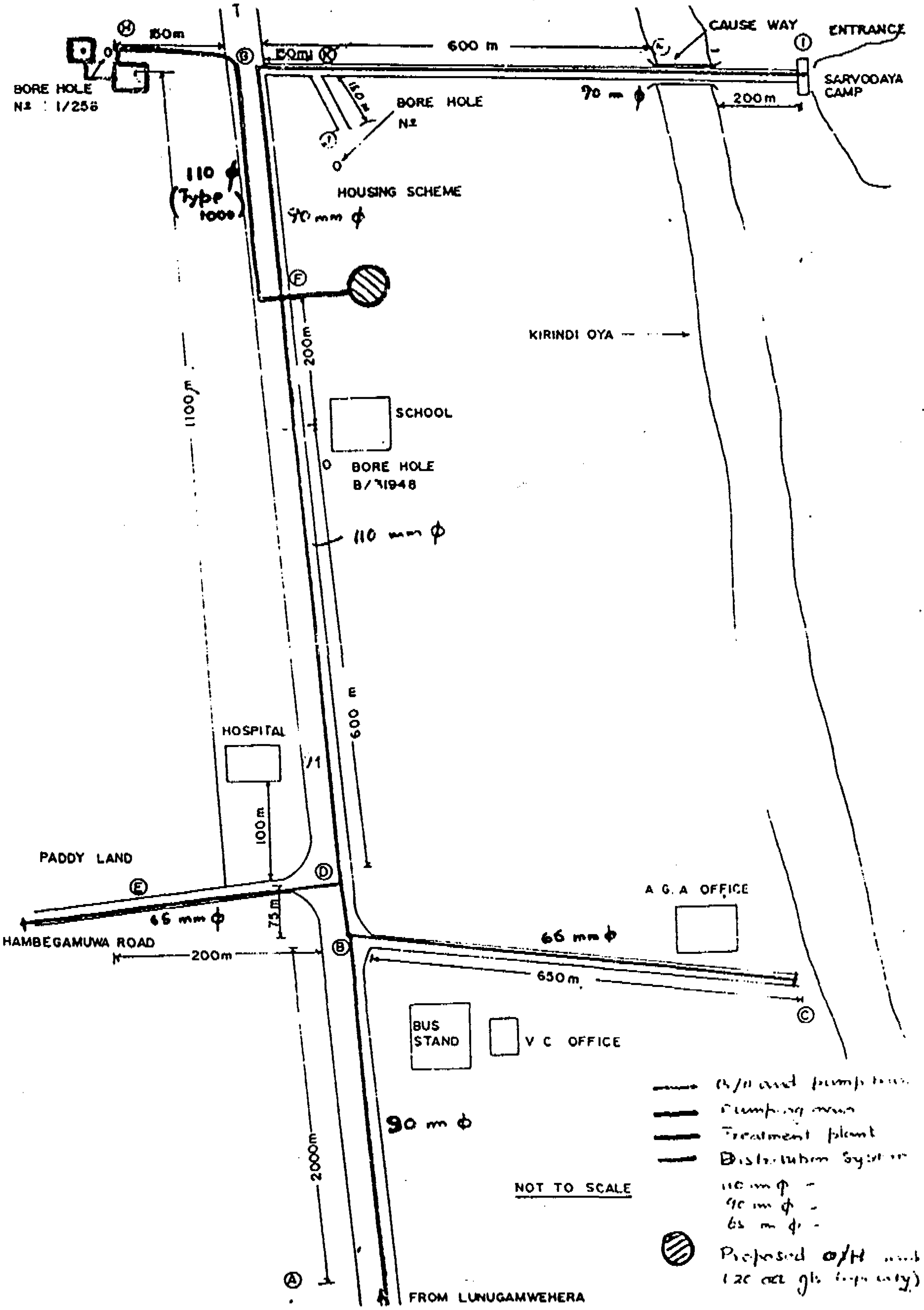


fig 5.1 Thanamalwila

5.2 Site investigations.

During a six week period (half of may - start of july, 1990), H.A.K.Amarakoon (Bsc, chemist at the NWS&DB Bandarawela) and R.J.Krol (student at the T.U.Delft in The Netherlands), investigated the treatment process in Thanamalwila.

At their first observation at the site, they found some lack of maintenance; a two cm thick layer of slime on the aerator and some algae drifting in the settlement tank. Out of the colour of the slime, the existence of a great number of iron bacteria could be concluded. They would certainly disturb the process of oxidation (as later occurred). After some turbidity tests, the aerator was "cleaned". At the same time, the V-notch weir was repaired and transformed to a 60 degrees V-angle (more accurate measurements as a 90 degrees V-angle).

In theory, the adding of chlorine, increased the process of transforming the soluble iron in non-soluble iron. This might have been a reason for the rapid clogging of the pipes:
The non-efficient aerated and settled water, still contained more iron than the desired level. After adding chlorine, most of the soluble iron was transformed into non-soluble iron complexes. Because of settling (especially in pipes with low velocities) and the forming of cristals, the iron particles attached themselves to the pipes as a nonremovable scale.
Therefore, the position of adding the chlorine was changed to a location right after the aerator; in front of the settlement tank. First a Belcom chlorinator was used (fig 5.3) and later on a bucket flow chlorinator (fig 5.4) was installed.

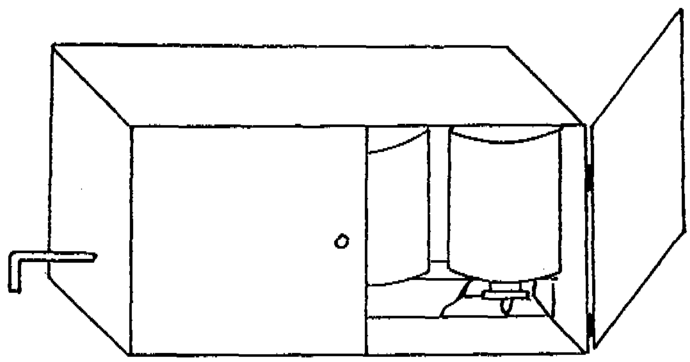


fig 5.3 Belcom chlorinator.

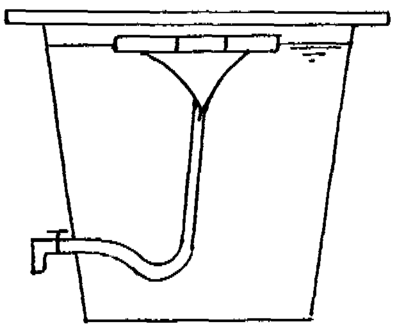
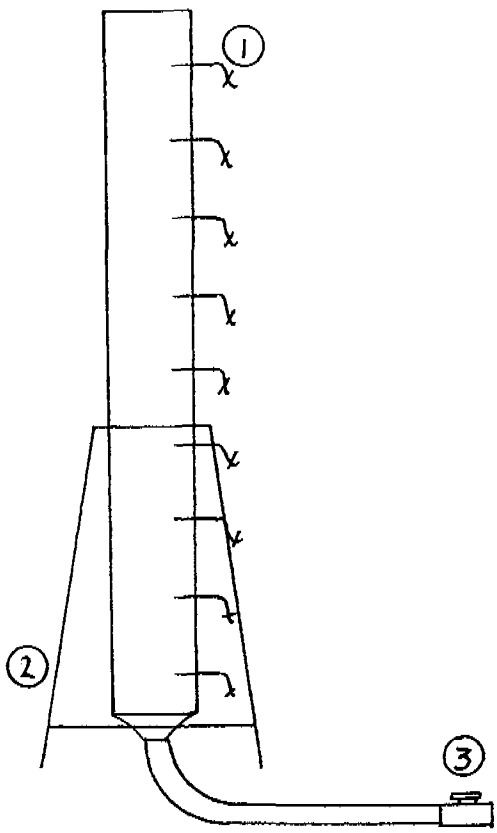


fig 5.4 A bucket flow chlorinator.

After a number of turbidity tests (in a 4 l. jar, during 15 min.), a settlement tube was constructed (fig 5.5) in order to get more detailed information of the settlement velocities in the settlement tank. Two 2.5 hour-sessions gave results as presented in appendix III.A and III.B. The samples were tested on turbidity as well as total iron content.



1) test tubes, closed by clips;

2) support bracket;

3) valve;

fig 5.5 Tube to determine settlement efficiencies.

The results of the settlement tests were discouraging: A maximum removal percentage occured of 40%. Because the iron content of the influent sometimes reached levels of 1.7 mg/l, it is not possible to reduce that content down to a level of 0.3 mg/l, the desired level. Therefore, even tilted plate settlers (fig 5.6) which probably could be installed in the present settlement tank, can do little to increase the efficiency of the settlement tank.

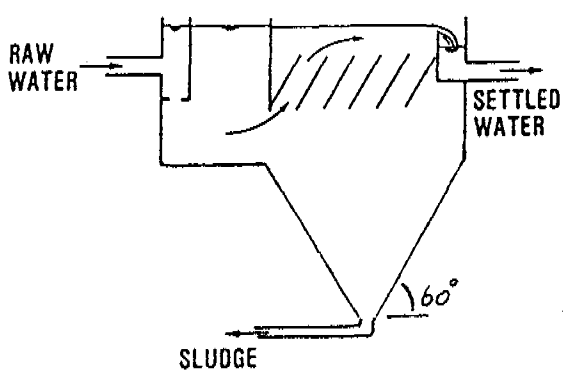


fig 5.6 Tilted plate settler.

5.3 Main Problems.

After site investigations and the information given by a report of the Pohonwala project, an irrigation project in the vicinity of Thanamalwila, the following problems can be summarised:

A) Waterquality:

1) Iron content.
This problem causes an enormous decrease of diameter of the distribution pipes; the end pipes with a normal diameter of 0.095 m. are reduced to a diameter of 0.072 m, which means a surface decrease of 42%. At the same time, some components of the distribution pumps have to be renewed every month.

2) Fluoride content.
In recent years, the fluoride content of the shallow wells caused problems to the teethcollor of young children. This is a well known problem with fluoride, but they occur just above a certain level (WHO standard: > 1.5 mg/l teeth problems, > 3.5 mg/l weakness problems of the bonestructure). For the moment the problem has been solved by the groundwelltube, but the high fluoride contents occurred only a few years ago at a well, which is located just outside todays watersupply scheme. Because the depth of the shallow well was allmost as deep as the tubewell's, the fluoride content has to be monitored thoroughly. (It is not clear why the design-section in Colombo choose for a tubewell, with the iron and fluorid problems, instead of using the river nearby. A possible answer is the high turbidity contents, which occur during the monsoons.)

B) Waterquantity:

1) Severe headlosses.
At the the ends uf the supplying lines, the pressure is not enough to supply a sufficient quantity of water. A temporary solution is provided by direct pumping of the highlift pumps to the end taps (in stead of a supply form the hospital tank). This actually means that the head, provided by the hospital tank, is not satisfying for the present situation.

2) No continuous supply.
Supplying hours are from 6 a.m. - 6 p.m. because of a severe drop in voltage afterwards. Because of this voltage drop, the highlift pumps are easily overheated. After 6 p.m, the hospital tank is able to supply for just 1 hour.

3) Unknown number of total demand.
Until now, the quantity supplied to the private taps, is limited to ca. 60% of the total. At present the total number of consumers (c.q. the Non Water Board schemes) is not known. Therefore, estimations of future demands, that

is future design of pipeline extensions, are difficult.

C) **Operation and Maintenance:**

1) **Education.**
The caretakers are not poperly educated. Because they do not understand the means of the technical equipment (aerator, settlement tank, chlorinator), the standard operation procedures are not of much use (the order in which the procedures have to be followed, is sometimes neglected). Maintenance is sometimes even left out (cleaning of aerator, settlementtank).

2) **Job satisfaction.**
Lack of payment and proper possibilities of promotion, don't do any good towards the motivation of the caretakers.

5.4 Possible solutions.

Because of these results, other possibilities have to be found in order to remove the non-soluble iron particles. Traditional solutions are:

1) Rapid sand filter: This is a very expensive solution, because of the sophisticated operation, special training for the caretakers is required. Furthermore, pumps are needed to backwash the system.
2) Adding chemicals: This needs a careful education (and control) of the caretakers. Because the chemicals have to be imported, in the long run, this solution will be expensive.

New solutions are (which means recently "redeveloped"):

3) Horizontal roughing filter: see recommendations for the Wellawaya site.
4) Upflow roughing filter: During the last years, a number of experiments have been carried out by the IRC in The Netherlands and the IRCWD in Switzerland with HRF's and URF's, which are less expensive and more easy to maintain and operate. Promissing results have been found.

Both options require however a pilot plant, in order to determine the exact dimensions and operation procedures.

5.5. Conclusions

The following conclusions involve only the technical side of the difficulties in Thanamalwila. These are made after a two month investigation, which has not ended yet, as is stated in the recommendations (5.6).
The other problems (operation and maintenance) should also be investigated thoroughly, because these might be more of a bottleneck towards the proper functioning of the system, as the technical ones.

A) Aerator:
It is clear that the thick slime layer, flourishing on the timber plates, disturbes the process of aeration and therefore the oxidation of the iron. This is the reason for a decrease of the turbidity, due to aeration.

B) Chlorinator:
The adding of chlorine is increasing the process of making the iron particles insoluble by transforming them into different complexes. Therefore, this should increase the iron removal. This process is also a cause for the clogging of the pipes, because of the adding of bleaching powder before distribution.

C) Settlement tank
Out of several tests of the tankefficiency itself (Appendix III.A) and the settlement of the particles of the intakewater (Appendix III.B), the efficiency of the tank seems to be zero. Therefore, the idea of improving the efficiency by means of a tilted plate settler (fig 5.6), appeared not to be sufficient.

Another possibile cause of a low efficiency, is the frequent disturbance of the process by distribution directly out of the settlement tank (at a random place) towards different institutions and towards the people outside the site-gate (at a watersupply site should be water, no?).

5.6. Recommendations.

Stage I.

A) Aerator:
The growth of this layer should be prevented by cleaning of the plates from time to time, say once a week.

B) Chlorinator:
At present, a primitive chlorinator is installed before the settlement tank. This should be transformed into a more sturdy (wind resistant) and better controlled installation.
The advised dosage is 1.0-1.5 mg/l (provided a settlement tank with a detention time of about 3 hours).
After rehabilitation of the iron remaval facility, the optimum dosage should be investigated again.

C) Settlement tank:
"Traditional" solutions towards the problem are the adding of

chemicals, coagulation and flocculation. But a less expensive answer may be the Horizontal Roughing Filter (as developed by Martin Wegelin; IRCWD Switzerland) or the Upflow Roughing Filter (as described by the IRC, The Hague, The Netherlands). In the long run, the latter could be less expensive because of the cheap non mechanical/chemical constituents needed and the low operation and maintenance costs.

These systems are originally designed to retain particles which cause immens turbidities, and can not be retained by settling basins. A side effect is the reduction of the total iron content and various other chemical constituents. It is assumed that because of the small size of the iron particles, gravity will not have much influence on the efficiency. Therefore, roughly the same results will be found with an upfow or horizontal filter. Because of this, only the upflow roughing filter will be investigated by building a pilot plant and operate it for a few months. For detailed information about the construction, operation and maintenance, monitoring and supervising programme, see appendix III.C.

In the mean time, proper operation and maintenance procedures should be provided to prevent the disturbance of the settling process. To prevent shortcutting, the inlet structure should be repaired.

Stage II.

A) Watersource:
After investigation of the possibilty to install a Horizontal Roughing Filter or Upflow Roughing Filter, a decision should be taken about the watersource: Is the present watersource satisfying or should it be moved to the Kirindi Oya, the nearby river. To take this decision, the raw water characteristics of the river should be investigated during a certain period (ideal is 1 year).

B) Water tower:
The design for the 90,000 l water tower, has already been made. Therefore the building should start immediately, if funds are available.

C) Extension of lines:
The rough outline of the extentension have already been prepared. Detailed design has to be done (elevation levels etc.).

6. References

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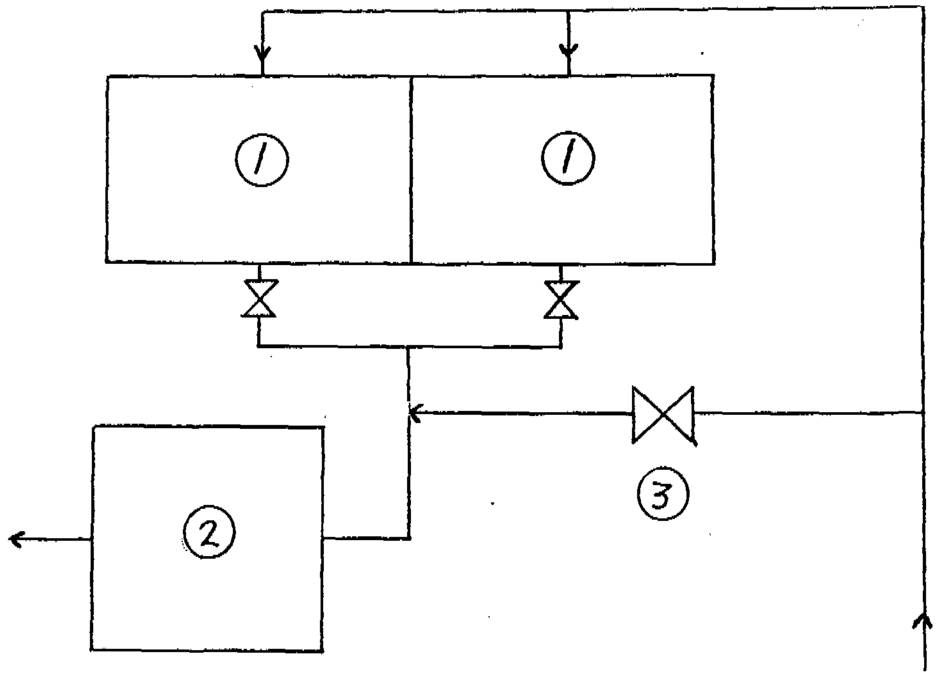
Appendix I : Wellawaya scheme

Appendix I.A : Site details
(figures out of a report of
the NWS&DB Bandarawela)



lay out diagram:
1) SSF 50' x 25'
2) 10,000 gls reservoir with chlorinator house
3) bypass

A. Design data 2 slow sand filters:

1) Average water demand in the year 2000	:	127000 gls/d
2) Maximum water demand in the year 2000	:	190000 gls/d
3) Assumed rate of filtration	:	0.2 m3/h/m2
4) Effective size of filter sand	:	0.3 mm
5) Uniformity coefficient of filter sand	:	> 2.5
6) Minimum filterthickness	:	0.7 m
7) Total iron content raw water	:	0.34 mg/l

B. Present demand

At the moment, the filters supply a 10.000 gal. reservoir at a rate of 150,000 gls/d, which is more as the designed rate for the present situation.
Supplying hours:

6.00 a.m. - 10.00 a.m.
4.00 p.m. - 8.00 p.m.

This means an average rate of 18750 gls/h.
The last sand suppletion (dec. 1987) consisted of 43 m3. Normally, scraping of the upper 2-3 cm, has to be done every month. During the monsoon, the water is distributed without filtering. If this bypassing would not be carried out, the filter would clog within a single day.

Horizontal Roughing Filter Wellawaya.

1. Introduction
2. Construction
3. Operation and Maintenance Procedures
4. Monitoring Program
5. Supervising Program
6. Monitoring Form

1. Introduction

During the rainy season, the turbidity of the river water rises tremendously because of the silt in the river, which settled down to the bottom during the year, resuspends again. When this water would be used as the raw water for a Slow Sand Filter, the filter would get clogged within a day or less.

The main purpose of this filter is to retain the small particles, the silt, which can not be removed otherwise (by a settling tank or tilted plates for example). Therefore, in most cases where this filter is operated, a settlement basin is designed to remove the heavy particles.

In the following design, only the HRF pilot plant is described. In the present situation it is not needed to construct a settlement basin for the HRF because the intake design functions as a kind of settlement tank. But in the future design, a settlement tank (or basin/tilted plates) will be needed. Therefore, it is recommended to think about the design of a settlement basin for the present situation (without HRF, only SSF) in order to reduce the turbidity and to shorten the period during which the non-filtered water is distributed.

The pilot plant consist out of a 13 m long tank with three filter compartments:

I : 2 m long, filtermaterial 3/4"-1/2",
II : 3 m long, filtermaterial 1/2"-1/4",
III: 2 m long, filtermaterial 1/4"-1/8".

These figures are concluded mostly from the experience of Martin Wegelin (loboratory tests) and the basic assumption of:
-turbidity filter influent water : 300 NTU,
-turbidity filter effluent water : <= 10 NTU.

Because future experience may indicate that the filter compartments should have a different length as the present, it is possible to remove the walls which divide the compartments and fixe them elsewhere. The designer should decide about this after one year (rainy season) has passed, information has been gathered and conclusions can be drawn.

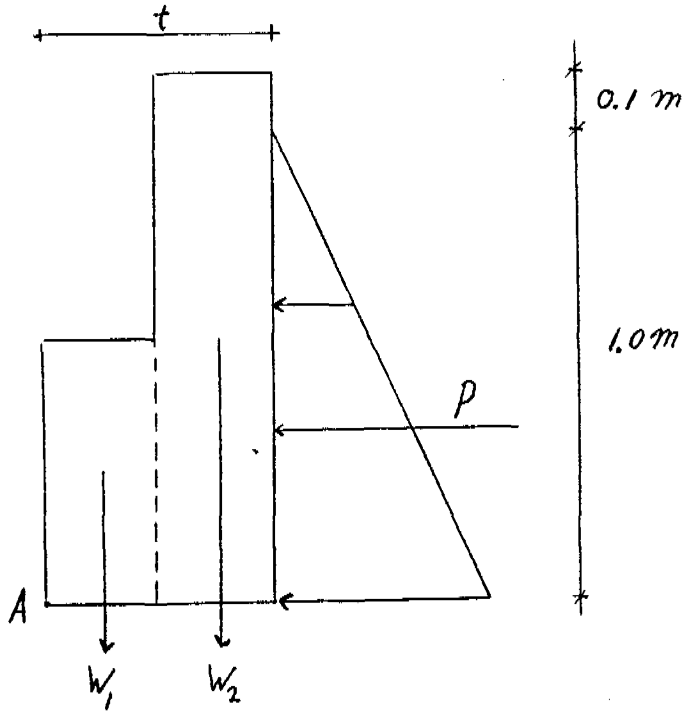
Errata

When the HRF design was completed, it was sent to the NWS&DB Bandarawela in order to start construction works (14/10/1990). Unfortunately, afterwards it appeared to be necessary to make some adjustments:

1) The outside wall thickness is calculated as follows:

Assumption: the chief break-down mechanism is the mechanism which tumbles the wall to its side.

Model:



The wall will not tumble around A when the momentum of P is less than the momentum of W1 and W2. What is the necessary thickness t.

$\sigma_{water} = 1000\ kg/m^3$, $\sigma_{brick} = 1800\ kg/m^3$, $g = 10\ m/s^2$

$P = ½ \times 1.0 \times 1000 \times 10 = 5\ kN/m^1$

$W1 = ½t \times 0.55 \times 1800 \times 10 = 4.95t\ kN/m^1$

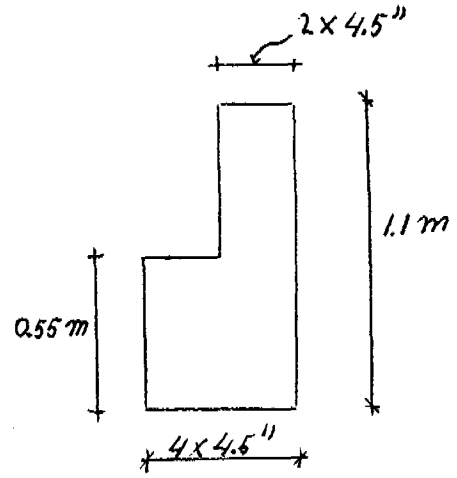
$W2 = ½t \times 1.1 \times 1800 \times 10 = 9.9t\ kN/m^1$

$\Longrightarrow P \times 0.33 - W1 \times 0.25t - W2 \times 0.75t <= 0$

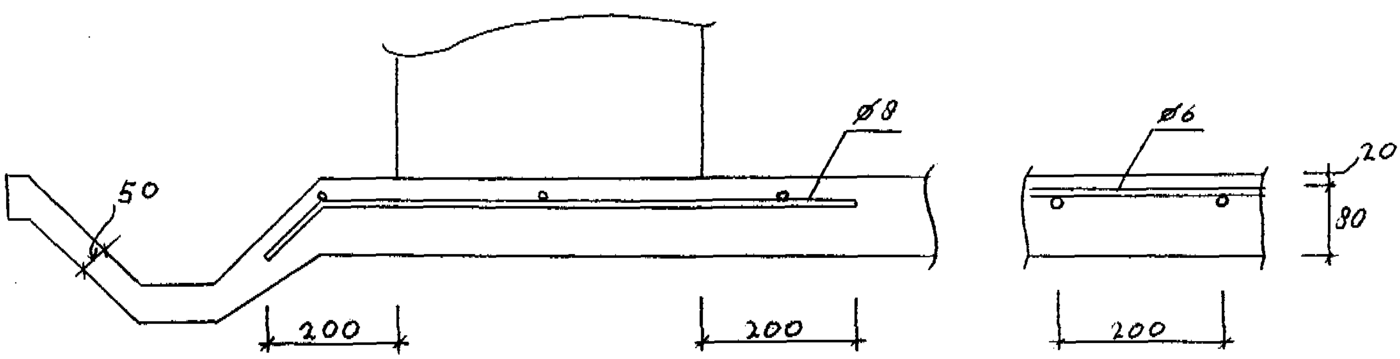
$1.65 - 1.24t^2 - 7.43t^2 <= 0$

$t >= 0.44\ m.$

==> take t = 4 x 4.5" bricks, which means;
bottom thickness = 4 x 4.5" bricks,
top thickness = 2 x 4.5" bricks.



2) The drainage along the foundation should be protected by a concrete slab, about 50 mm thick. This is needed to protect it against erosion during the wash out. It is recommended to fasten this drainage to the foundation in order to prevent the water from undermining the foundation. Because of this, some minor reinforcement, 8 mm diameter x 200 mm width, is needed for the concrete:



Distances in mm.

2. Construction

Because there is some time left for constructing the filters, I assumed that there is enough construction material (or money) available to build the pilot plant in "more suitable" dimensions, in respect of the width of the filter compartments (0.8 m instead of 0.4 m) and the wall thickness.

Overall system.
The raw water enters the filtersystem at the balancing tank, where water pressure is reduced.
Out of this tank, it enters the inlet by a ball-valve, which controls the level. By a small tube, about **200 mm** below the water level the water enters the distribution compartment. This has to be fixed at a discharge of **0.2 l/s** by adjusting the hight of the ball-valve, which fixes the waterdepth.
Because of the holes in the seperation walls, the water is distributed homogeneously throughout the filter compartments:
- Compartment I: filtermaterial 3/4"-1/2", length: 2 m,
- Compartment II: filtermaterial 1/2"-1/4", length: 3 m,
- Compartment III: filtermaterial 1/4"-1/8", length: 2 m.

After reaching Compartment IV (empty), the water overflows at 0.9 m above the bottom into the outlet compartment. Through the drainage pipe, the water can directly be flushed out.

Balancing Tank.
This tank, a clean watertank normally used for temporary provisions, is needed to reduce water pressure: see fig. 1.1. To maintain a certain pressure head, the reservoir should be installed upon the reservoir of the nearby Slow Sand Filter. In this way it is possible to use a ball-valve in order to get the right discharge (0.2 l/s).

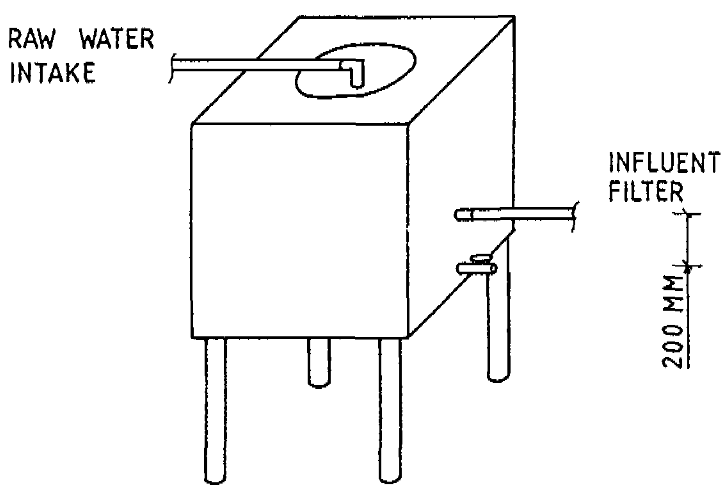


fig. 2.1 Balancing tank.

A side effect of using this tank, is the accumulation of sediment at the bottom of the tank, like in a settlement tank. Because of this, provisions should be made to clean the tank and to prevent the intake of the filter from clogging. This can be done by installing a second valve, about 200 mm above the bottom of the tank, as an outlet to the filter with the ball-valve. The valve at the bottom (already available), can be used to remove the sludge after a certain period. If it is necessary, the reservoir can even be cleaned by stopping the filter operation and putting the reservoir up side down (not preferable).

Foundation.

Because of the inner width of the compartments (800 mm), the wall thickness (9" at the bottom) and the piers (9" out of the upper wall), the foundation has to have a width of:
$800 + 2x9x25.4 + 2x4.5x25.4 = 1500$ mm.
This means, the existing foundation has to be widened up to 1500 mm.

To make it more easy to work in a compartment, a minimum width and length of 800 mm should be taken. Therefore it is better to increase the filter length up to 13 m. This means an increase of 1 m. It does not matter at which side the foundation is lengthened. Only the slope of 1:100 should be kept in mind.

The material should be the same: non-reinforced concrete of about 100 mm thick (about 1").

Walls.

The wallthickness, except for the separation walls, should be 9" up to half of the hight (about 550 mm). Above this hight, 4" should be sufficient. The total hight of the walls (above the foundation) is 1100 mm.

Piers of 9"x9" should be made, which means 4.5" out of the wall at the bottom and 9" out of the wall at the top.

Separation walls should be made at a 4.5" thickness. The walls A, B, C and D should be "open" in such a way that 20% of the area is covered by holes. This can be achieved by not putting any plaster in the vertical clefts between the bricks, each with a width of 15 mm. The walls B, C and D should not be fastened to the outer walls. This to make future adjustments more easy.

Except for the "open" separation walls, the inside of the filter should be plastered entirely in order to make the tank watertight.

Intake.

The intake should be built in the same way as the rest of the filter, except for the height. This should be 1300 mm, in order to maintain a certain head over the filter.

An 8 mm diameter plastic tube should be put 350 mm below the top of the inlet, through the wall at the distribution compartment.

The ball-valve should be adjusted in such a way that a constant discharge through the 8 mm plastic tube into the distribution compartment of 0.2 l/s is reached. The position of this valve can be estimated as 200 mm above the level of the plastic tube.

A gauging-rod should be installed in the distribution compartment. In order to measure the zero-, clean- and stop-level during operation.

Outlet.

After the water has passed compartment IV, the water overflows into the outlet compartment. By the drainage pipe of the bottom, the water can be flushed out. Care should be taken not to let the water flow around the foundation, because it might decrease the bearing capacity of the soil. Therefore, a small ditch should be excavated and protected by some plastic foil. See fig. 2.2.

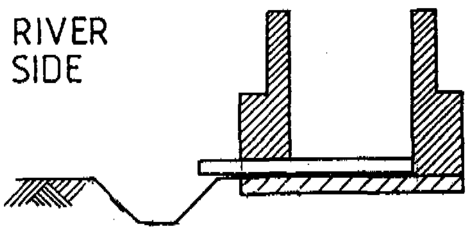


fig. 2.2 Drainage along foundation.

Drainage (detail A).

For a quick washout of the filter, 3" pvc pipes are needed at the bottom of the filter compartments. As described in the HRF manual of M. Wegelin p. 44, 6 mm diameter holes should be made at two sides (opposite to eachother) at a spacing of 100 mm. This is just over a length of 800 mm (the width of a compartment). The total length of the pipe should be 1500 mm. See detail A in the drawing.

The pipes can be closed by the already prepared caps (see drawing), made out of the wide end of 3" pipes, by melting the non-fitting side together. The caps should be put gently at the pipe ends in order to remove the caps more easily when needed.

The outlet of the pipes should be situated at the river side of the filter. This to prevent the water from flowing along the foundation.

Test Tubes (detail B).

At the compartments I, II and III (filled with filter material), test tubes to measure the turbidity should be provided. They should be placed in the middle and at the end of a compartment (100 mm and 700 mm above the bottom). This means every compartment has 4 test tubes (noted as t; see the longitudinal section and detail B in the drawing).

Flood Protection.

Because of the rainy season, measures should be taken to protect the construction against the water running down the hills, along the Slow Sand Filter and the HRF. Possibilities are:

- excavation of a ditch,
- a small earthen wall.

3. Operation + Maintenance Procedures

a) Start filter operation only when the compartments are completely filled with the filter material. If not, the efficiency will be poor.

b) Before starting filter operation, wash the installed filter material by drainage.
   - Fill the filter with a low flow rate (0.5 - 1.0 m/h) up to the effluent level.
   - Discharge the water through the drainage pipe, located nearest to the inlet.
   - Repeat this procedure 2 or 3 times.

c) Intermittent operation is possible under the condition of a "smooth" start. This means filling the filter again at a slow flow rate (0.5 - 1.0 m/h).

d) The flow rate at the inlet should be controlled daily and kept constant at 0.2 l/s.

e) Operation of the filter should be directed by the total headloss. The total headloss can be measured at the inlet gauging rod:

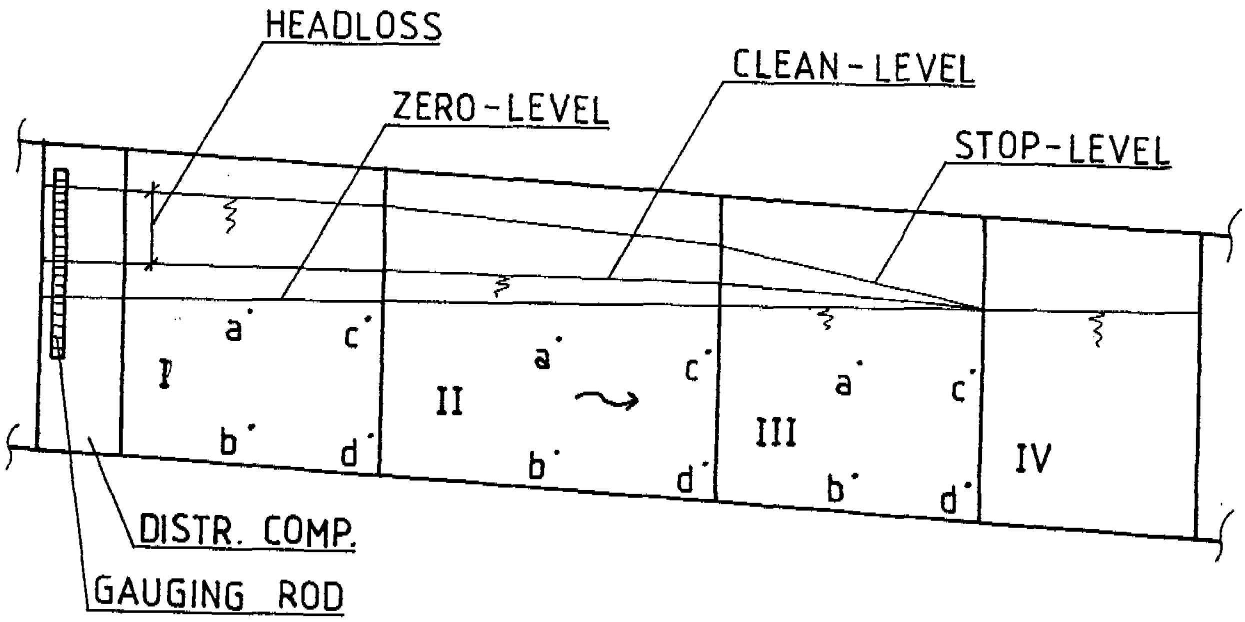


fig. 2.1 Waterlevels at different stages of operation.

1) When you start using the filter, measure the level at the inlet and at the back of every compartment (I, II and III), when the water is standing still; this will be called the <u>zero-level</u>. This can be executed by using the tubes for the turbidity measurement as a piezometer (see detail B at the drawing),

2) Operate the filter at the required rate and measure the inlet level again; The difference with the former level is the total headloss for a clean filter. This level will be called the clean-level,

3) At the same time, measure the waterlevels at the downstream end of compartments I and II. The difference between these levels (the clean-levels) and there zero-levels gives the headlosses over the different filter compartments at this stage (see fig 2.1),

4) When operating the filter, the filter material will get clogged, especially during the rainy season (within 2-3 weeks). This causes a rise in the headlosses over the filter compartments, which results in a rise of the total headloss.
At a certain time, the operation of the HRF has to be stopped; this will be called the stop-level. This moment has arrived when **one of the next two** criteria has been achieved:

- the intake water level has increased up to **100 mm** above the clean-level; this should be the stop-level,
- the effluent turbidity has increased to > 10 NTU (provided a Hach turbidity meter is installed). The stop-level should be the level 20 mm below this intake level.

The lower one of both levels should be fixed as the stop-level.

5) At that time, measure the water levels at the back of compartments 1 and 2. The difference between these levels (the stop-levels) and their clean-levels, gives the headlosses over the compartments due to clogging of the filter material (see fig 2.1),

f) Hydraulic filter cleaning:
Start the cleaning procedure at the intake side:
1) Open drain 3, for full drainage (only 1 drain functions),
2) Open drain 4, for full drainage (only 1 drain functions),
3) Open drain 5 and 6, for full drainage (only 2 drains function simultaneously),
4) Open drain 7,8,9 and 10, for full drainage (only 4 drains function simultaneously).

Initially it is advised to repeat the procedure at least for 4 times. When, after thorough cleaning and smooth restarting of the filter, the total headloss (to be measured at the inlet, as said before) has only decreased to a level of 2 cm below the stop-level, the filter has to be cleaned manually,

g) Manual cleaning:
In case of a clogged filter (see par. 2.f) or just before the start of the rainy season (hence once a year), especially before peak loads are occurring, the filter should be cleaned manually. This means:

- Excavating the filter material from the drained filter compartments, starting with the coarsest filter material,
- Washing of the filter material by "mechanical" stirring in a washwater basin. This can easily be done by using sieves which only permit the water to pass. For example, this can be done in the nearby river or by using the bypass which sprays the water over the sieves. By friction, the impurities will be removed.
  Take care **not to break** and **not to mix** the filter material, otherwise the material has to be sieved again (on particle size)!
- Re-installation of the filter material should be done immediately after washing, to avoid contamination with dust or other impurities. Disintegrated material must be replaced. Therefore it is advisable to have a stock of additional filter material, kept at the site.

i) Never keep the HRF dry unless the filters are properly cleaned in advance. Otherwise a slimy layer may form a skin around the filter material, which hinders the efficiency.

4. Monitoring Programme

a) The flow rate has to be kept at 1.0 m/h (or 0.2 l/s), which has to be adjusted every day by the caretaker.

b) The turbidity has to be measured every week, but at periods of high turbidity, every day. This has to be done at every "measuring tube", with a Hach turbidity meter (NTU) and put to paper at the prescribed form.

c) The total headloss (difference between "absolute" in- and out-let level) and compartment headlosses have to be measured at the same rate as the turbidity and put to paper at the same form.

5. Supervising Program

A monthly report should be made and sent to the above address in The Netherlands. In order that contact be maintained and advise given if need be.
This report should include information about:
- turbidity,
- headlosses,
- hydraulic cleaning,
- operation in general.

When the pilot program has ended, an advise will be given for the final dimensions.

Because of lack of information, it is not possible to give exact figures about the time of hydraulic and manual cleaning. After experimenting with the filter and deciding about the periods of hydraulic cleaning, it is advisable to have a last hydraulic cleaning just before the high turbidity period arrives at the filter. The phases of the below described supervising program, are only a rough estimation of the time which is really needed. These should be experienced.

Phase 1: 1 week.
After starting the filter, at least one week is needed to get acquainted with the filter and its basic operations. In this period the supervising engineert should take samples every day at the test tubes to test the turbidity and to note the headlosses. In other words: He has to follow the monitoring program.

At the same time he has to teach a caretaker, how to operate the filter. If the filter is functioning in good order, actually nothing has to be done by the caretaker except for the testing procedures (turbidity and headlosses). It is important to start instructing the caretaker at the beginning of this phase, in order to check his way of dealing with the testing procedures during this phase.

Phase 2: until high turbidity period starts.
During this phase, only the caretaker is looking after the site and executes the testing procedures. It is recommended to have a site visit by the supervising engineer at the end of every week. The engineer should have a look if the stop-level has been reached. If this is the case, the filter has to be cleaned hydraulically.

By this weekly check the engineer should be able to make a final decision about the period whereafter hydraulic cleaning is necessary. The time of hydraulic cleaning can be set in two ways:

1) When the stop-level is reached, hydraulic cleaning should take place,
2) After a certain time (when some headlosss has been acquired but not yet the stop-level), for example 2 weeks, the filter should be cleaned hydraulically.

The choice depends on the capability of the caretaker.

Phase 3: high turbidity period.
During this period, information should be gathered about the time to clean the filters manually. This really depends on the filter efficiency and the efficiency of the hydraulic cleaning. It is possible, the time of manual cleaning occurs at a time the high turbidity period is not over yet. Because it is not useful to have a manual cleaning during the high turbidity period, it is recommended to continue operation until this period has passed.

When the rainy season is over, final conclusions can be made about the filter dimensions.

6. Monitoring Form Wellawaya.

week number	
date of start	
supervising engineer	

Only first day of first monitoring week and after hydraulic cleaning:

day	headloss of clean filter (mm) (difference zero- and clean-level. Ia, Ib, etc refers to fig. 2.1 of the O+M procedures)												
	distr. comp.	I a	b	c	d	II a	b	c	d	III a	b	c	d
1													

day	distr. comp.	turbidity (NTU) I a	b	c	d	II a	b	c	d	III a	b	c	d
1													
2													
3													
4													
5													
6													
7													

day	difference clean-level and present level (mm) distr. comp.	I a	b	c	d	II a	b	c	d	III a	b	c	d
1													
2													
3													
4													
5													
6													
7													

Appendix I.C : HRF design details.

This design is made according to the guidelines provided by Mr. M.Wegelin, in his HRF manual. He did some investigations at different HRF's under laboratory conditions with a kaolin suspension; a suspension to simulate the raw water characteristics.

The objective of an HRF design is the reduction of the suspended solids in the raw water. The raw water characteristics determine the filter lay-out and operation. The required capacity only determines the cross-section area of the filterbed. So for the design of a pilot plant, it is not necessary to know the future capacity, although a certain figure is required to decide about the feasibility.

The following four design variables determine the HRF lay-out:
1) filtration rate v_f in m/h, which is the hydraulic load in m^3/h on the filter's cross-section area in m^2,
2) the individual sizes d_{gi} of the filter material in mm,
3) the individual lengths L_{fi} of each filter material in m,
4) the cross-section area A of the filter in m^2.

Assumptions:
1) the kaolin suspension used by Mr. Wegelin, more or less equals the performance of the raw water.
2) the initial raw water turbidity C_i = 500 NTU. This can be reduced down to a level of 300 NTU by a settling tank; C_0 = 300 NTU,
3) the available filter material has gravel sizes of 3/4", 1/2" and 1/4". The performance of this material is assumed to be the same as material with the size of 20, 10 respective 5 mm; the average sizes Mr. Wegelin worked with.

Mr. Wegelin investigated the performance of different HRF's under different flow speed conditions. This resulted in tabel 1 and 2 with some "tentative" design guidelines and efficiency values for each compartment.

Tabel 1: the values are given for loads in mg/l. However, the same guidelines are valuable for the same values in NTU.

Tabel 2: after choosing a v_f, the L_{fi} should be determined with the method of trial and error, in order to reach the desired efficiency.

maximum suspended solids concentration in presettled water	C_o (mg/l)	>300 high	300-100 medium	<100 low
filtration rate	v_F (m/h)	0.5	0.75 - 1	1 - 1.5
filter length for	l_f (m)			
d_g = 20 mm		3 - 5	3 *)	3 *)
15 mm		2 - 5	2 - 4	2 - 3
10 mm		2 - 4	2 - 3	2
5 mm		1 - 2	1 - 2	1
maximum suspended solids concentration in HRF effluent	C_e (mg/l)	5	2 - 3	2

*) this gravel fraction can possibly be omitted

Tabel 1: Tentative design guidelines.

$$E = \frac{C_e}{C_o} = e^{-\lambda \cdot l_f} \quad [\%]$$

Gravel Size d_g	Filtration Rate v_F [m/h]	Filter length l_f [m] 1	2	3	4	5
5 mm	0.5	15.2	2.3	0.4	0.1	0.
	0.75	28.3	8.0	2.3	0.6	0.
	1	39.9	15.9	6.4	2.5	1.
	1.5	59.0	34.8	20.5	12.1	7.
	2	74.7	55.7	41.6	31.1	23.
10 mm	0.5	35.6	12.7	4.5	1.6	0.
	0.75	50.7	25.7	13.0	6.7	3.
	1	61.7	38.1	23.5	14.5	9.
	1.5	77.7	60.3	46.9	36.4	28.
	2	89.5	80.2	71.8	64.3	57.
15 mm	0.5	48.4	23.5	11.4	5.5	2.
	0.75	62.4	39.0	24.3	15.2	9
	1	72.1	51.9	37.4	27.0	19
	1.5	85.4	72.9	62.2	53.1	45
	2	95.0	90.2	85.6	81.3	77
20 mm	0.5	56.9	32.4	18.4	10.5	6
	0.75	69.6	48.5	33.7	23.5	16
	1	78.1	61.0	47.6	37.2	29
	1.5	89.5	80.1	71.7	64.2	57
	2	97.7	95.4	93.2	91.0	88

Tabel 2: E-values for HRF.

Considerations:

1) From tabel 1 can be concluded to use a v_f of 0.75-1.0 m/h,
2) The experience of Mr. Fellinga, reference nr. 5, chapter 6, shows some other tentative design guidelines; The highest absolute efficiencies can be achieved at the first compartment. The length of this compartment does not

contribute much (a relative efficiency increase of 18-26 %, depending on the length). The absolute efficiency of the next compartments is much lower (abs. eff. of 10-14 %), but the contribution of the length is much larger (rel. eff. increase of gravel filters; 25-70 %, ijuk filters (a kind of kokos fiber); 44-200 %). These figures are strongly depending on the raw water characteristics,

3) With the experience of Mr. Fellinga and Mr. Wegelin, it can be said that the total length of the filter should be 7-9 m. Initially, 3 compartments seem enough, given the beforementioned length estimation,
4) For a pilot plant it should be possible to change the length of the compartments after the initial design has been monitored for a year.
5) Design rate: maximum demand in year 2000: 190,000 gls/d = 8000 gls/h (with an operation of 24 h/day).

Iteration procedure:
1) take a velocity v_f 0.75-1.0 m/h;
2) estimate the length L_i of the 3 compartments;
3) conclude with the aid of tabel 2 (absolute compartment efficiencies E_i if the turbidity can be reduced from C_0 = 300 NTU down to C_e <= 10 NTU;
4) calculate the total length L_T ;
5) repeat this procedure until a satisfactory value of L_T and C_e have been achieved.

Calculations:
(v_f in m/h, L_T in m, C_e in NTU)

1) v_f= 0.75	L_1 =2 E_1=0.39	L_2 =4 E_2=0.067	L_3 =3 E_3=0.023	L_T =9	C_e =0.18
2) v_f =1.00	L_1 =2 E_1=0.519	L_2 =4 E_2=0.145	L_3 =3 E_3=0.064	L_T =9	C_e =1.44
3) v_f =1.00	L_1 =2 E_1=0.519	L_2 =4 E_2=0.145	L_3 =1 E_3=0.399	L_T =7	C_e =9.00
4) v_f =1.00	L_1 =3 E_1=0.374	L_2 =4 E_2=0.145	L_3 =1 E_3=0.399	L_T =8	C_e =6.50
5) v_f =1.00	L_1 =3 E_1=0.374	L_2 =3 E_2=0.253	L_3 =1 E_3=0.399	L_T =7	C_e =10.5
6) v_f =1.00	L_1 =3 E_1=0.374	L_2 =3 E_2=0.253	L_3 =2 E_3=0.159	L_T =8	C_e =4.19
7) v_f =1.00	L_1 =2 E_1=0.519	L_2 =3 E_2=0.253	L_3 =2 E_3=0.159	L_T =7	C_e =5.80

Because of the above calculations and considerations, the following filter will be chosen:

Compartment	Length(m)	Filter material (")
1	2	3/4-1/2
2	3	1/2-1/4
3	2	1/4-1/8

Determining the height H of the filter:
Given the design rate of 8000 gls/h = 36.4 m^3/h;
v_f = 1.0 m/h. This means A = 36.4 m^2.
The maximum width of a filter is about 4 m. The total length for the filter is estimated at 10 m because of the intake/ outlet constructions and the possibility of a 9 m long filter. This information can give a rough estimation of the required area.
What will be the width W?

H = 1.0 m ==> W = 36.4 ==> 9 filters of 4X10 = 360 m^2
or an area of 20X18 m.
H = 0.9 m ==> W = 40.4 ==> 10 filters of 4X10 = 400 m^2
or an area of 20X20 m.
H = 0.8 m ==> W = 45.5 ==> 11 filters of 4X10 = 440 m^2
or an area of 20X22 m.
H = 0.6 m ==> W = 61 ==> 15 filters of 4X10 = 600 m^2
or an area of 20X30 m.

Because the available space is not much of a problem in the future, an height of 0.9 m is chosen.

For further information about details in the design is referred to the manual of M.Wegelin.

Appendix II. Welimada scheme: Site details

Technical data:

- center of the main tube (diameter about 4 m, depth about 30 ft.) about 8 m from the riverside,
- 3 laterals: D= 6 inch, 87 holes (D= 0.01 m) p. feet,
- 2 pumps (6000 gls/h each). In the morning, 1 pump takes 5 hours to empty the well; in the evening it takes 4 hours. After pumping, the well is refilled within 4 hours,
- reservoir: 36.000 gls. (is situated about 200 m. above intake). Chlorine is added by hand; in general 1 kg p.day, in times of high turbidity 1.5 kg p.day.

Appendix III : Thanamalwila scheme

Appendix III.A : Research details.

Present situation:

- 1 well (no. 1/256), 1 pump (7.5 H.P, Qmax = 20.000 l/h, Qave = 11.250 l/h, Htot = 42.1 m, borehole depth = 36 m),
- Aerator: 2 m. above the ground is a tank constructed; on a tank of 1 m hight, are 6 square timber plates (2.3 x 2.3 m.) constructed with ca. 0.45 m space between them,
- Distribution to the settlement tank: after a V-notch weir (which is not functioning at the moment), the water passes a ca. 4 m long, 0.34 m wide, open canal,
- Settlement tank: L = 7 m, H = 3.32 m, W = 1.8 m,
- Reservoir: Volume 5000 gls. Chlorination by bleaching powder),
- Elevation house with 2 pumps (elevates the water to an other reservoir,
- Total demand: 60.000 g.p.d.
- All is built in reinforced concrete.

Main problem:
Iron contend varies of 0.5-1.5 mg/l,
(maximum desirable level: 0.3 mg/l,
maximum permissable level: 1.0 mg/l).

Investigation:

A) **General:**

1) No chlorine has been added at the reservoir (by a Belcom chlorinator) since january (today is end of may), because this increases the oxidising process in the distribution pipes and therefore in the clogging proces.
2) 14 may 1990.
   Stand post at the site:
   - Coliform bacteria : 370.
   - E-coli bacteria : 30.
3) Quite a lot of algae were flourishing in the small canal discharging to the settlement tank.

B) **Aerator:**

1) The aerator is not functioning all the time. This is because the 5.000 gallons reservoir is quite soon filled, while the two reservoir pumps are quickly overheated and have to be stopped now and then.
2) On top of the aerator is a thick layer of slime growth of about 1 cm. This slime is flourishing due to lack of maintenance and intermitted functioning of the aerator.
3) They stopped maintenance of the aerator, because of the bad condition of the timber plates.
4) 14 may 1990.
   Water before aeration:
   - turbidity 10.1 NTU.
   Water after aeration :
   - turbidity 3.9 NTU.

These figures are contrary to what was expected; a rise in turbidity due to aeration (oxidation of the Fe particles).

5) 30 may 1990.
   Water before aeration:
   - 0.35 NTU. The turbidity increased during the jar test up to 1.0 NTU (because of the detention time, needed to oxidise the Fe).
   Water after aeration :
   - 1.5-2.0 NTU.
6) 21 juni 1990 (after thoroughly cleaning of the aerator).
   Bypass water:
   - 0.8 NTU. (distilled water 1.0 NTU, no reference).
   - 0.7 mg/l Fe.
   Top of aerator:
   - 0.65 mg/l Fe.

C) Chlorinator:

1) A bucket flow chlorinator has been installed between the aerator and the settlement tank.
2) 30 may 1990.
   Without chlorinator:
   - 1.5 mg/l Fe.
   After chlorination:
   - 0.5-0.7 mg/l Fe.
   - 1.0 mg/l Cl (constant).
   Start of settlement tank:
   - 1.5 NTU. The turbidity remained at a constant level, during a 15 minute jar test. Normally the turbidity should decrease, but because of the processes of aeration and chlorination, which increase the turbidity by producing non-soluble Fe, it did not change.
   End of settlement tank:
   - 0.2-0.3 mg/l Cl.
3) 21 june 1990 (after thoroughly cleaning of the aerator).
   After chlorination:
   - 0.55 mg/l Fe.
   - 1.7 mg/l Cl.
   End of settlement tank:
   - 0.3 mg/l Fe.
   - 1.5 mg/l Cl (due to a temporary stop of the aerator while the chlorinator kept running, the water was contaminated with an overdosage).

D) Settlement tank:

1) 30 may 1990.
   Start of settlement tank:
   - 1.5-2.0 NTU. Intake water as wel as outlet water of the settlement tank, were initially very low. The reason for this might be the intermittent aerator pumping.
   End of settlement tank:
   - 2.1 NTU. The turbidity of the water passing the settlement tank, did not change if not to say

increase. The efficiency of the settlement tank seems to be zero.

3) 15 june 1990.
   Start of settlement tank:
   - turbidity: A settlement test was carried out during 2.5 hours; water taken from the tank after 1 hour detention time. After 1.5 hours of decreasing turbidity, the turbidity started to rise again; up to a level of the starting value or even above. Although the water was not homogeneous (range 3-4.5 NTU), this does not explain the rise of turbidity for all the samples, which were taken in a range of 1.4 m (height of the tube is 2 m).
   - Fe content: One of the tubes during the settlement test was tested during 2.5 hours. The Fe content decreased from 0.5 to 0.3 mg/l.
   - Conductivity of the water: 1030 US/cm.
4) The process of settling is disturbed by:
   - frequent distribution towards different institutions, who just plug in their distribution line somewhere at random in the settlement tank, and the nonregistered supply to the tap at the gate.
   - shortcircuiting, due to dismal functioning of the inlet structure.

Appendix III.B : Settlement efficiency charts.

The settlement velocity of the precipitated particles, was measured by a 2 m, 4" tube. The tube was perforated by 9 copper tubes of 5 mm diameter, 200 mm hart to hart, to get a sample at a certain time and hight of the tube. Samples were taken at T = 0, 1, 3, 5, 8, 10, 15, 25, 40, 60, 90, 120 and 150 minutes, to measure the turbidity as well as the total iron content.

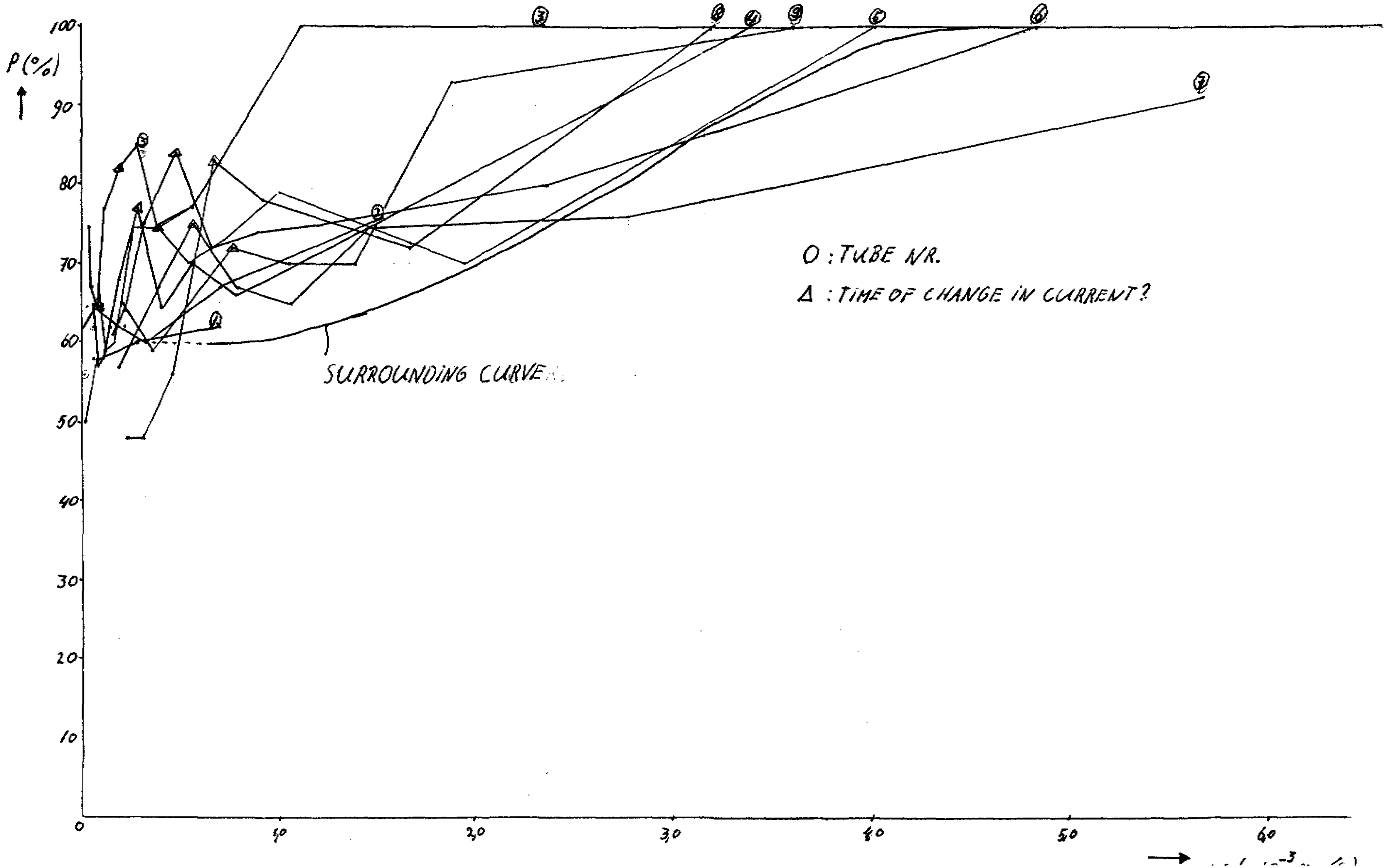
By deviding the height (h = distance to the top water level) by the time t (in seconds because the timing was not always correct), a certain velocity v (m/s) is achieved. All particles with an higher velocity, are below the sampling point (assuming an homogeneous water volume). By measuring the turbidity or total iron content, the removal percentage (p = sample/initial turbidity or total iron content * 100) can be calculated.

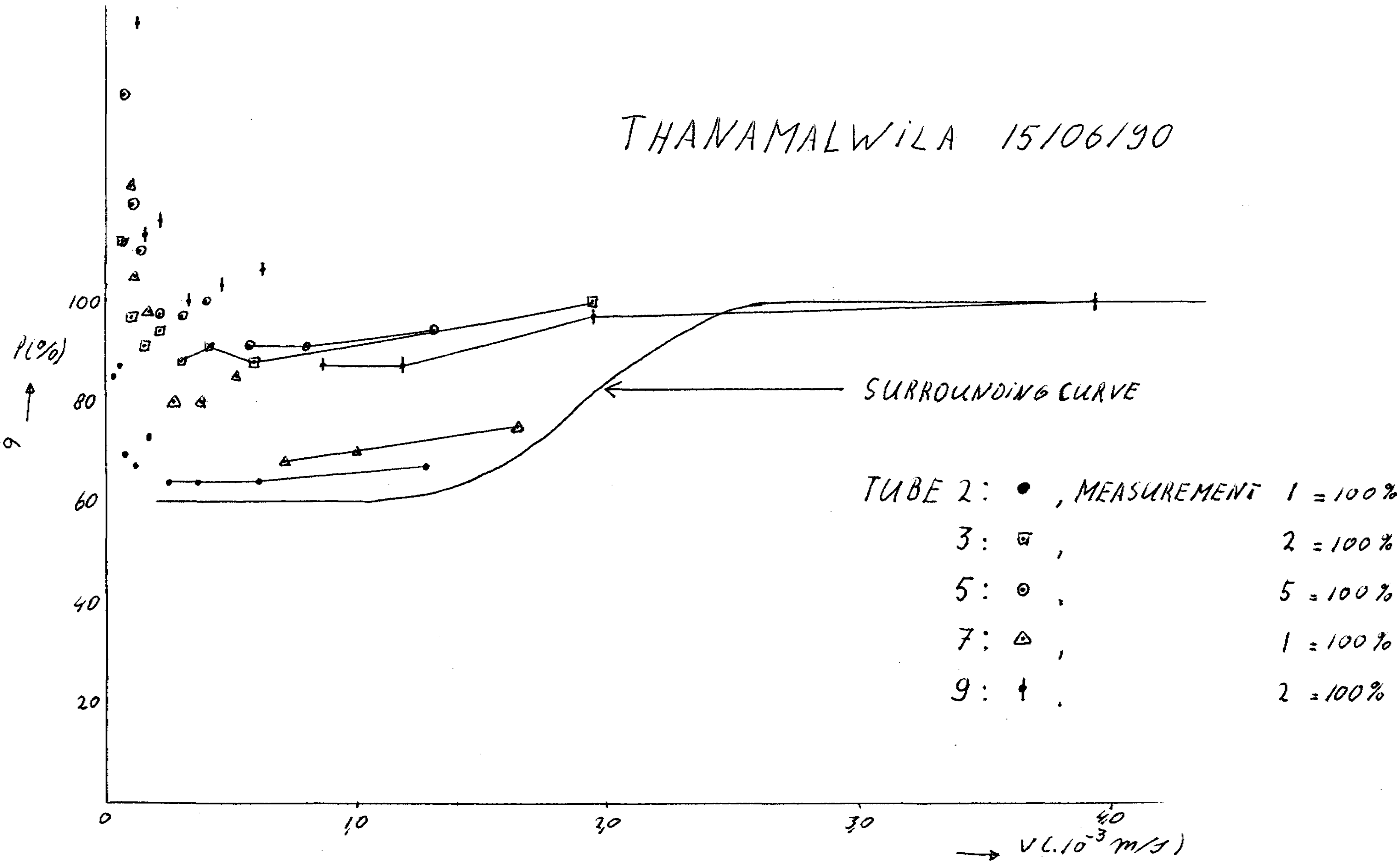
It appeared to be a problem to get a homogeneous sample. Therefore, the samples of 12/06/90 and 15/06/90 show great differences. Despite all this, even when a very pessimistic curve is drawn (the surrounding curve), it can be concluded that the maximum efficiency will not exceed to more than 40 %.

Another (minor) detail about the measurement at 12/06/90 is that a certain time a sudden increase of turbidity occurred at all sample tubes. This may be explained by a sudden change in current, which caused a different measurement with the Hach turbidity meter.

Because of the surrounding curves, it can be concluded that just 40% of the suspended solids will finally, after a long time, settle. This efficiency is not enough. Therefore, even a tilted plate settler, which is a concept based upon the process of settling, will not satisfy the requirements (at least 85%).

THANAMALWILA 12/06/90 , 14.00 H.




THANAMALWILA 15/06/90
SURROUNDING CURVE
TUBE 2: ●, MEASUREMENT 1 = 100%
3: ⊡, 2 = 100%
5: ⊙, 5 = 100%
7: △, 1 = 100%
9: ⍿, 2 = 100%
P(%)
100
80
60
40
20
0
10
20
30
40
V (.10^-3 m/s)

Appendix III.C : URF pilot plant manual

Upflow Roughing Filter Thanamalwila

1. Introduction

The main purpose of this filter is to retain the small, oxidised, iron particles. This constituent has to be reduced down to a level of between 0.3 mg/l (advisable) and 1.0 mg/l (maximum).

At the moment the system consists out of an aerator, chlorinator and settlement tank (see drawing 1). Even supposed the system is maintained in a proper way (clean aerator, right dosage of chlorine and a homogeneous, laminar flow through the settlement tank), the system will not function because of the small size of the particles. Therefore a roughing filter should be installed. Because of the low turbidity and the small size of the particles, only two compartments are investigated. Gravel size:
1) 1/2" - 1/4"
2) 1/4" - 1/8"

To reduce costs, the possibility will be investigated of installing the filter in the (non-functioning) settlement tank. Therefore, the pilot plant will be operated at the same rate as the rate, which is allowable for the settlement tank area. It is estimated that two URF's of 3x1.8 m can be installed in the existing tank. Because of this assumption, the pilot plant will get the vertical dimensions of the settlement tank (c.q. the hight of the filterbed).

At the moment, the system is operated intermittently, because of a shortage of current and storage. Suppose the proposed filter can be installed in the present settlement tank, the filter should be tested for the actual situation. However, the settling process will increase the efficiency and disturb the normal situation after the filter operation has been stopped. Therefore it is thought better to operate the filter non-intermittent. The intake is situated in a corner of the aerator "reservoir", where the water is not influenced by the chlorinator. The aerator reservoir has enough storage to overcome the time the aerator is not functioning. A positive side effect of this decision is that the time, necessary for the operation of the filter, is shortened.

2. Construction

Overall system.
See drawing 2. The URF consists out of 2 PVC tubes of 8" diameter, with a height of 3 m, which can be operated seperately or together. **The top of the filter tubes is at +250 mm (250 mm above the bottom of the aerator reservoir = 0000 mm)**. The pipes are filled with the filter material over a height of 2.5 m:
filter tube 1: 1/2"-1/4",
filter tube 2: 1/4"-1/8".

The intake is a syphon, made by 1/2" PVC pipes. The open end is situated 100 mm above the bottom of the aerator reservoir. Filter 1 is fed, at its bottom by a 1/2" PVC pipe. After flowing upwards through filter 1, the water exits the first filter by a 1/2" PVC pipe. At the bottom of filter 2, the water enters again by a 1/2" PVC pipe. After flowing upwards through filter 2, the water exits again through a 1/2" PVC pipe.

Drainage of the system can be executed at the filter outlets (1" PVC pipes), located at the bottom of the filter tubes, at the other side of the inlets.

The filter should be operated at a rate of 0.0282 l/s (is ca. 17 l in 10 minutes), which is controlled by a water meter and a valve between the intake and filter 1.

Intake at the aerator reservoir.
The intake is located in a corner of the reservoir, not to be influenced by the chlorinator. It consists out of a 1/2" PVC pipe and has its open end at +100 mm, to prevent clogging because of the accumulated particles at the bottom. A syphon over the edge of the aearator reservoir, discharges the water to the tube intake at the bottom of filter 1 or 2.

Valves.
In order to operate the filters separately, 6 valves are needed to direct the water into the right tubes:
3 1/2" valves and 3 1" valves.

To clean the filter manually, the valve and pipe system should be untied from the filter tubes. Therefore the valves should not be fastened by glue to the pipes.

Test tubes at the filter.
Test tubes should be installed at the following locations:
a) filter 1: at the intake of the tube (use a 1/2" T-socket and an adjusted 1/2" PVC pipe, see fig.A),
b) filter 1: 950 mm above the intake,
c) filter 1: 450 mm below the top of the tube (same height as the outlet to filter 2),
d) filter 2: at the intake of the tube (use a 1/2" T-socket and an adjusted 1/2" PVC pipe, see fig A),
e) filter 2: 950 mm above the intake.

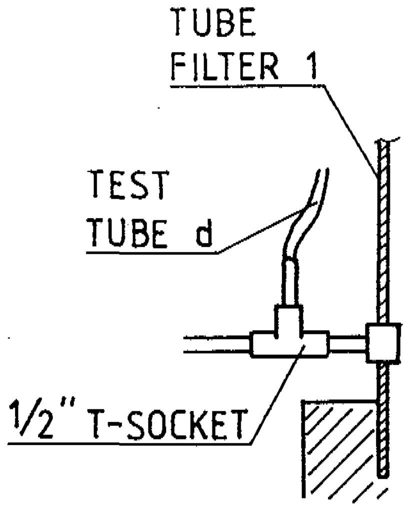


fig. A: test tube at the intake
of a filter tube.

Test tubes b, c and e, can be installed by using a 6 mm diameter copper pipe, 100 mm long. For 50 mm, the test tube should reach into the filter material. It is recommended to use Arraldite to glue the test tubes at the filter tube.

By fixing transparant, plastic tubes at the test tubes up to the level of the top of the filter tube (see fig.B), the piezometrical level can be measured in order to calculate (total) headlosses.

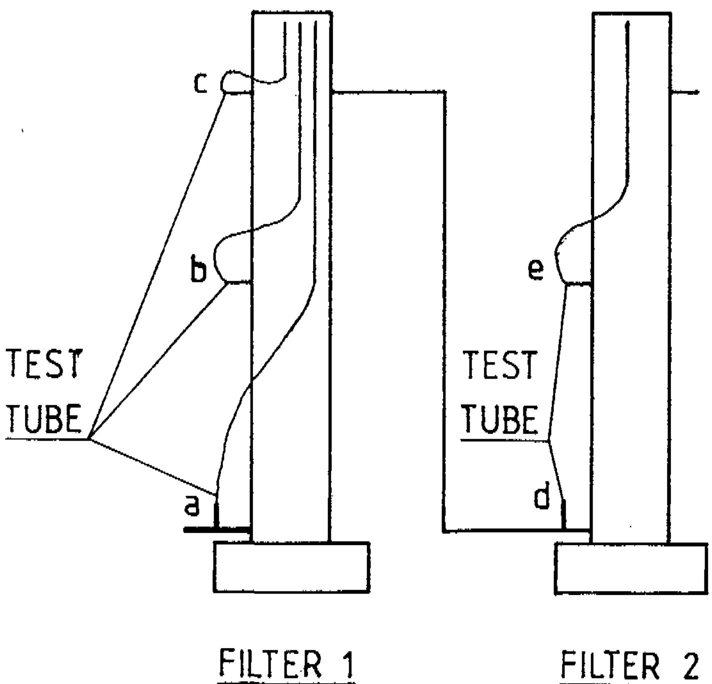
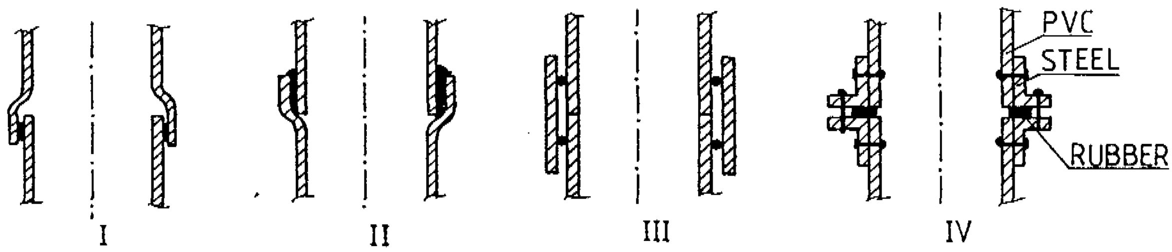


fig. B: plastic tubes, fastened
to the filters.

Foundation of the filter tubes.
In order to simplify the manual cleaning and to be able to check the intake and outlet construction at the inside, the tubes should be cut into 2 halves at 450 mm from the bottom of the tube. During operation, the tube halves are fastened watertight to eachother by joints. Depending on the local



JOINTS:

situation and experience, one can choose one of the joints (I, II, III or IV) presented at the drawing. The important things are to keep the filter watertight during operation and the possibility to untie the tube halves.

To maintain a constant flow through the filter, it is very important to keep the top of the filter tube below the level of +250 mm. When the top of the filter is above this level, it could be difficult to maintain a constant discharge, due to lack of pressure. Therefore, the bottom of the filter tube should be below -2750 mm.

In order to get watertightness at the bottom, the tube bottoms should be put into 100 mm concrete. See drawing.

Intake/outlet at the filter tubes.

Detail A: Because of the hydraulic cleaning, retained particles will accumulate at the bottom. To avoid resuspension, the intake should be 150 mm above the level of the outlet. Because of the concrete, the filter outlet is located 100 mm above the bottom of the tube. At this position, a 1" diameter hole is tapped in which a 1" screw socket can be installed. Opposite the outlet, the intake is constructed 250 mm above the bottom of the tube. At this position, a 1/2" diameter hole is tapped in the tube wall, in which a 1/2" screw socket can be installed.

At the inside of the tube, 2 small "walls" of bricks and/or concrete should be made to support a steel plate of the same size as the inner diameter of the fiter tube. The hight of the "walls" is 200 mm (above the concrete of the foundation). The distance between them is 100 mm.

Between the "walls" some concrete should be pasted with a small angle of inclination. The purpose of this is to make it more easy to remove the remaining particles at the bottom and to prevent accumulation below the intake.

Detail B:

The steel plate (thickness >= 2 mm) should be perforated with 4 mm diameter holes, at intervals of 20 mm.

Iron strings to support the filter tubes.

Because the wind can put some pressure on the tubes and therefore move them, it is necessary to fasten them at about 1000 mm below the top by 3 iron strings. A strip of iron should be fastened around the tube, at which the strings can be tied. Because of the manual cleaning (removing and cleaning the filter material), it is necessary to roll the tubes. Therefore it should be possible to untie the iron strings.
The other end of the strings should be fastened to some concrete blocks in such a way that no movement of the filter tubes is possible.

3. Operation + Maintenance Procedures

The filter will be operated continuesly and can be operated in 3 different ways:
I : operation of filter 1 and 2,
II : operation of filter 2,
III: operation of filter 1.
Normal operation is case I. Case II will be investigated at a later stage. Case III is an option, only when case II is not possible. In the following o+m procedures, a specification will be given for the different cases.

a) Start filter operation only when the compartments are entirely filled with the filter material up to a level of 50 mm below the top outlet (is 500 mm below the top of the filter tube).
All cases the same procedure.

b) The syphon can start to function by sucking water through a tube, located just at the site where the water enters the system through the water meter.
All cases the same procedure.

c) At the first day of filter operation, wash the installed filter material by drainage (=hydraulic cleaning):
   1) Fill the filters with the required (low) flow rate (17 l in 10 minutes) up to the effluent level.
      Case I : Close valves: 1, 3, 5 and 6,
      Open valves: 2 and 4,
      The flow rate can be adjusted by valve 2,
      Case II : Close valves: 2, 3, 5 and 6,
      Open valves: 1 and 4,
      The flow rate can be adjusted by valve 1,
      Case III: Close valves: 1, 3, 4 and 6,
      Open valves: 2 and 5,
      The flow rate can be adjusted by valve 2,
   2) Discharge the water through the 1" outlet at the bottom.
      Case I : see case II and III,
      Case II : Close valve 1, 2, 3, 4 and 5,
      Open valve 6 (as quickly as possible),
      Case III: Close valve 1, 2, 3, 4 and 6,
      Open first valve 5 and thereafter, as quickly as possible, valve 3,
   3) Repeat this washing procedure 3 times.
      All cases the same procedure.

d) The flow rate at the intake should be controlled daily and kept constant at a rate of 17 l in 10 minutes. By using a stopwatch and measuring the flow rate at the outlet in a bucket, the caretaker can check and calibrate the watermeter.

e) The filter should be cleaned hydraulically every day at the same time (at the end of the day for instance).
Before cleaning, the turbidity, total iron content and headlosses have to be measured.
After hydraulic cleaning (see (c), only one time flushing), the filters have to be filled again and started smoothly.

Turbidity, total iron content and headlosses have to be measured again. The first half hour, the filter needs to establish its function again of retaining particles. During this period, the water will not be cleaned because the filter is not functioning in a proper way; the water has to be thrown away. After this time, again turbidity, total iron content and headlosses have to be measured.

f) Operation of the filter can be directed by the headloss. The total headloss can be found by measuring the difference between the piezometric water level at the intake of the filter(s) and the outlet level of the filter(s):
Case I : Intake test tube a, outlet filter 2,
Case II : Intake test tube d, outlet filter 2,
Case III: Intake test tube a, outlet filter 1,

1) When the caretaker starts using the filters, the level at every test tube, should be measured first when the water is standing still; this will be called the zero-level. These levels should be marked at the filter tubes.
All cases the same zero-level,
2) Operate the filter at the required rate and measure the waterlevels at all the test tubes. The difference between these levels and there zero-levels will be called the clean-levels. These give the headlosses over the different filter areas at this stage. These levels should be marked at the filter tubes.
The clean-levels will be different for the different cases. These cases should be marked seperately at the tubes,
3) While operating the filter, the filter material will get clogged. This causes a rise in the headlosses over the filter areas, which results in a rise of the total headloss.
At a certain time, the operation of the URF has to be stopped; this will be called the stop-level. This moment has arrived when **either one of the next two** criteria has been achieved:

- the inlet water level has increased up to 100 mm above the clean-level; this should be the stop-level,
- the total iron content of the effluent turbidity has increased up to a level of 1.0 mg/l. The stop-level should be the level 2 cm below this inlet level.

The lower one of both levels should be fixed as the stop-level. In any case, the total iron content of the effluent should be <= 1.0 mg/l.

All cases will have a different stop-level, which should be marked separately.

5) At that time, measure the water levels at the other test tubes. The difference between these levels (the stop-

levels) and their clean-levels, gives the headlosses over the filter areas due to clogging of the filter material. These levels should be marked.
All cases will have a different stop-level, which should be marked separately.

f) Hydraulic filter cleaning:
This procedure can be executed in the same way as (c). Only one flushing is necessary. When, after thorough cleaning and smooth restarting of the filter, the total headloss has only decreased down to a level of 2 cm below the stop-level, the filter has to be cleaned manually.
Total headloss:
- Case I : piezometric level difference tube a and outlet filter 2,
- Case II : piezometric level difference tube d and outlet filter 2,
- Case III: piezometric level difference tube a and outlet filter 1,

g) Manual cleaning:
When the stop-level has been reached, the filter should be cleaned manually. This means:
1) Unloosen the inlet and outlet pipes at the valves,
2) Untie the iron wires from their concrete blocks,
3) Untie the filter tubes (in a gentle manner) from their joints,
3) Remove the filter material from the drained filter and put it on sieves. Take care not damage the test tubes,
4) Wash the filter material by "mechanical" stirring in a washwater basin. This can easily be done by using sieves which only permit the water to pass. By spraying the water over the sieves and by stirring the filter material on the sieves, the impurities will be removed by friction.
   Take care **not to break** and **not to mix** the filter material, otherwise it is necessary to sieve (on particle size) it again.
5) Re-installation of the filter material should be done immediatly after washing, to avoid contamination with dust or other impurities. Disintegrated material must be replaced. Therefore it is advisable to have a stock of additional filter material, kept at the site.

i) Never keep the URF dry unless the filters are properly cleaned in advance. Otherwise a slimy layer may form a skin around the filter material, which hinders the efficiency.

4. Monitoring Programme

(All cases same programme)

a) Every day at the same time, the filter(s) have to be cleaned hydraulically and restarted again.

b) The flow rate has to be kept at 17 l in 10 minutes (0.0282 l/s), which has to be adjusted every day by the caretaker after restarting the filter.

c) The turbidity has to be measured three times every day:
   1) Before hydraulic cleaning,
   2) Immediat after restarting the filter,
   3) 30 minutes after restarting the filter.
   This has to be done at every test tube, with a Hach turbidity meter (NTU) and put to paper at the prescribed form.

d) The same samples, and the same frequency as noted under c) can be used to determine the total iron contents by the Hach kit. This has to be put to paper at the prescribed form.

e) The total headloss (see (f) of the O+M procedures) has to be measured every day before the hydraulic cleaning takes place. This has to be put to paper at the same form.

f) The waterlevels in the test tubes have to be noted at the same rate as the total headloss and be put to paper at the prescribed form.

5. Monitoring Form Thanamalwila

Form Case I:

week number	
date of start	
supervising engineer	

headloss clean filter (mm)	
-test tube a to b	
-test tube b to c	
-test tube c to : d	
-test tube d to e	
-test tube e to outlet	

day	turbidity (NTU)						total iron (mg/l)					
	a	b	c	d	e	outlet	a	b	c	d	e	outlet
1 1												
2												
3												
2 1												
2												
3												
3 1												
2												
3												
4 1												
2												
3												
5 1												
2												
3												
6 1												
2												
3												
7 1												
2												
3												

day	headloss (mm)					
	a-b	b-c	c-d	d-e	e-outlet	total
1						
2						
3						
4						
5						
6						
7						

Form Case II:

week number	
date of start	
supervising engineer	

headloss clean filter (mm)	
-test tube d to e	
-test tube e to outlet	

day		turbidity (NTU) d	e	outlet	total iron (mg/l) d	e	outlet
1	1						
	2						
	3						
2	1						
	2						
	3						
3	1						
	2						
	3						
4	1						
	2						
	3						
5	1						
	2						
	3						
6	1						
	2						
	3						
7	1						
	2						
	3						

day	headloss (mm) d-e	e-outlet	total
1			
2			
3			
4			
5			
6			
7			

Form Case III:

week number	
date of start	
supervising engineer	

headloss clean filter (mm)	
-test tube a to b	
-test tube b to c	
-test tube c to outlet	

day		turbidity (NTU) a	b	c	outlet	total iron (mg/l) a	b	c	outlet
1	1								
	2								
	3								
2	1								
	2								
	3								
3	1								
	2								
	3								
4	1								
	2								
	3								
5	1								
	2								
	3								
6	1								
	2								
	3								
7	1								
	2								
	3								

day	headloss (mm) a-b	b-c	c-outlet	total
1				
2				
3				
4				
5				
6				
7				

6. Supervising Programme

A monthly report should be made and sent to the above address in The Netherlands. In order that contact be maintained and advice given if need be. This report should include information about:

- turbidity,
- total iron content,
- headlosses,
- hydraulic cleaning,
- operation in general.

When the pilot programme has ended, an advise will be given for the final dimensions.

Because of lack of information, it is not possible to give exact figures about the time of hydraulic and manual cleaning. The phases of the below described supervising programme, are only a rough estimation of the time which is really needed. These should be experienced.

The described phases have the same estimation of time needed for every case (as described in the O+M procedures). One has to start with case I (both filters) and thereafter case II (filter 2). Case III (filter 1) only has to be investigated in case of problems occurring at case I.

Phase 1: 1 week.
After starting the filter, at least one week is needed to get acquainted with the filter and its basic operations. In this period the chemist should take samples every day at the test tubes to test the turbidity, the iron content and the headlosses. In other words: He has to follow the monitoring programme.

At the same time he has to teach a caretaker, how to operate the filter. If the filter is functioning in good order, actually nothing has to be done by the caretaker except for the testing procedures (turbidity, iron content and headlosses). It is important to start instructing the caretaker at the beginning of this phase, in order to check his way of dealing with the testing procedures during this phase.

Phase 2: ca. 7 weeks.
During this phase, only the caretaker is looking after the site and executes the testing procedures. The chemist has to make a site visit at the end of every week. The chemist should have a look if the stop-level has been reached.

During this period, information should be gathered about the time to clean the filters manually. This really depends on the filter efficiency and the efficiency of the hydraulic cleaning. It is possible, the time of manual cleaning already occurs after one month, although this is not probable. Another possibility is that clogging will occur just after half a year or even longer. But after two months or less, it should be possible to make final conclusions.

When final conclusions have been drawn about case I, the filters have to be cleaned manually. Therafter, the whole procedure has to start again with case II.

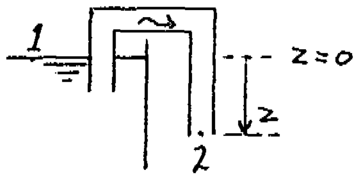
Appendix III.D : URF design details

Most detail designs are based upon practical considerations and need no explanation. However two details have to be discussed: the required discharge and the required head.

A) The required discharge: The design is based upon the assumption that it should be possible to install two filters of 3x1.8 m in series in the existing settlement tank. Therefore it is needed to know the actual pumping rate. Assumed that this is a constant rate, this can be measured by taking the height over the crest of the V-notch. With the discharge curves, presented in ref.nr.9, see chapter 6, the discharge can be determined: Q = 4.7 l/s
For a pilot plant filter of 8" diameter, this means a discharge of:

$$4.7 \times \frac{\frac{1}{4}\pi(8 \times 0.0254)^2}{3 \times 1.8} = 0.0282 \text{ l/s is about 17 l/10 minutes}$$

B) How much head is needed for a discharge of 17 l/10 minutes? Assume a tube hanging with one end in the water and the other end outside.



1. z=z, p=0, v=0

2. z=0, p=0, v=?

Without friction, the discharge is determined by the differences in piezometric level between the ends of the tube:

Bernoulli: $\frac{p}{\sigma g} + z + \frac{v^2}{2g} = \text{constant}$

end of tube: $z = \frac{v^2}{2g} \Longrightarrow v = \sqrt{2gz} \Longrightarrow Q = A \times \sqrt{2gz}$

$$= \tfrac{1}{4}\pi d^2 \times \sqrt{2gz}$$

$$\Longrightarrow (4Q/\pi d^2)^2 \times 1/2g = z$$

With Q = 17 l/10 minutes = 2.83×10^{-5} m^3/s,

$d = 1/2" = 0.0127$ m,

$g = 9.81$ m/s^2 ,

$\Longrightarrow z = 0.0025$ m

This is almost nothing. Assume for the total required head H = 0.1 m; because of friction of the tubes, bendings and valves.

B) During the night, the water level will drop because the pumps stop to work. During the next 12 hours, the aerator reservoir has to supply 17 l/10 minutes = 1.23 m^3.

The surface area A of the aerator reservoir = 9.45 m^2.
The water will drop during the night with 0.13 m.

C) During the process, the filter will start to clog. The porosity will reduce and the filter resistance will increase. To overcome this friction, a certain head is required. The required head can be calculated with the formula of Carman Kozeny:

$$I_0 = 180 \times \frac{\mu}{g} \times \frac{(1 - P_0)^2}{P_0^3} \times \frac{v}{D_0^2}$$

with I0 = initial slope of pi zometric surface in the filter-bed,
μ = kinematic viscosity (m^3/s) at a certain temperature,
P_0 = initial or present porosity,
v = rate of filtration (m/s),
g = co fficient of the gravity acceleration (m/s^2),
D_0 = diameter of the filter material.

Present situation:
μ = 0.9 x 10^{-6} m^2/s for a temperature of 25°,
g = 9.81 m/s^2,
P_0 = 0.3 - 0.4,
v = Q/A
Q = 17 l/10 minites

= 2.83 x 10^{-5} m^3/s

A = 0.032 m^2

= 8.73 x 10^{-4} m/s,
D_0 = 0.005 m (1/4" - 1/8").

P_0 = 0.30,	I_0 = 0.01 m,	required head H = I_0 x L = 0.025 m.
0.35	0.006	0.015
0.40	0.003	0.008

So the initial bed resistance is about 0.025 m for one filter, which means about 0.05 m for both filters.

D) When the intake of the filter is located 0.1 m above the bottom of the aerator reservoir and the outlet of the second tube is located 0.2 m below the aerator reservoir bottom level, the available head will be:

+0.5 (water level in reservoir) - (-0.2) (outlet level) - 0.13 (night decrement) - 0.1 (friction) - 0.05 (filter friction) = +0.47 m.

This head is available for the clogging process:

==> about 0.23 m for each filter ==> I_0 = 0.092

==> what is P? With trial and error and Carman Kozeny:

==> P = 0.16

==> When the porosity has decreased from 0.35 down to 0.16, the filter will stop functioning. This is satisfactory.