67

Operational Efficiency Of Conventional Iron Removal Plant In Municipal Water Works Of Bangladesh

by

Shamsul Gafur Mahmud

A thesis submitted to the Department of Civil Engineering of BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, (BUET) Dhaka in partial fulfillment of the requirement for the degree

of

MASTER OF SCIENCE IN CIVIL ENGINEERING

1999

OPERATIONAL EFFICIENCY OF CONVENTIONAL IRON REMOVAL PLANT IN MUNICIPAL WATER WORKS OF BANGLADESH

by

Shamsul Gafur Mahmud

A thesis approved as to style and content for the degree of M.Sc. Engineering (civil)

Farooque Ahmed

Dr. Farooque Ahmed — **:Chairman** (Supervisor)
Professor
Department of Civil Engineering
BUET, Dhaka

Dr. Md. Humayun Kabir — **:Member**
Professor and Head
Department of Civil Engineering
BUET, Dhaka

Dr. Md. Delwar Hossain — **:Member**
Professor
Department of Civil Engineering
BUET, Dhaka

Md. Quadir-uz-Zaman — **:Member (External)**
Chief Engineer
Department of Public Health Engineering
Government of the People's Republic of Bangladesh

DECLARATION

I hereby certify that the research work reported in this thesis has been performed by me and this work has not been submitted elsewhere for any other purpose (except for publication).

December, 1999

(Shamsul Gafur Mahmud)

Table of Content

List of Tables

List of Figures

Acknowledgement

The author is grateful to Allah, the almighty and the merciful who enable the author to carry out the study and complete the thesis successfully.

I am grateful to Dr. Farooque Ahmed, my supervisor and professor of civil Engineering, BUET, for his continuous support, advice and guidance. He introduced me to this fascinating world of research in water treatment. He, not only guided me but also helped me to keep patients during the study.

My thanks to Mr. Md. Quadir-uz-zaman, Chief Engineer, DPHE, for his support to carry out the study. He allowed me to go to the field for carrying out field investigations. He has special attention to his officers for professional development through study and training. I also bothered him many times for my matters during his busy schedule of official work.

My special thanks to Dr. Md. Delwar Hossain, professor, Department of Civil Engineering, BUET, for his encouragement and valuable advice in carrying out the study and also in improving this thesis.

My thanks are also for Dr. Md. Humayun Kabir, Professor and Head, Department of Civil Engineering, BUET for his overall direction and for providing his valuable time for this research work.

I would also like to thank Mr. Shahjahan Mallic, Mr. Ibrahim, Mr. Mizanur Rahman, Mr. Mofazzal, Mr. Ihteshamul Haq and other colleagues for their sincere cooperation and providing valuable information.

Lastly my hearty and sincere gratitude to my mother and father who always encouraged me to study more and more and prayed for me for my success.

Abstract

In Bangladesh about ninety percent of municipal water works are using ground water as a source of drinking water. The reason is this water requires no treatment for bacterial contamination and can be supplied directly. But ground water contains minerals like iron, manganese and arsenic etc. of which iron is most commonly present in ground water of this country.

The presence of iron in ground is generating various complications in municipal water supply system. This complication is not limited to technical aspect but also to the management of municipal water works. The horizon of this complications due to this iron is increasing as the number of thana towns are upgraded to municipalities.

In the past, inadequate measures have been taken to minimize the iron problem through installation of limited number of iron removal plants. Due to resource constraints installation of IRPs in large number could not be made possible. But with the number of municipalities the demand for removal of iron from water is growing. During eightees, the Netherlands government came forward to minimize the problem and installed a number of IRPs. But several operational difficulties showed up within short period of commissioning. In course of time the plant performances are reported to decline and their O&M cost stands so high that it becomes very difficult for the municipal authority to meet these cost out of their revenue income.

In this study the declining performances of the IRPs were investigated. The study also examined the effectiveness of various steps of the treatment process and their findings were analyzed along with observational, experimental and theoretical findings. During the study it was found that these IRPs were not designed on the basis of any pilot plant performance or on the basis of any laboratory model test.

Study also reveals that plant performances are declining due to some design faults. Design of Wash trough, filter underdrain and determination of backwash rate in accordance to grain size are worthwhile to mention.

Poor backwash creates a couple of problems : the impurities are not completely removed at one hand and on the other hand premature filter breakthrough occurs which means water quality deteriorates much before filter head lodd reached to terminal head. It is found that no plants could meet neither the designed water quality nor WHO guideline value for drinking water.

In this study, a laboratory model test was performed in order to determine the unit processes of treatment that would require for the study area on the basis of the investigation. A unit process of treatment is selected and design outline is given on the basis of the model test.

Some operational procedures are suggested, as observed during the study. If these were implemented, plant performances would improve by 30 to 50% with respect to wash water requirement. Moreover some modifications are also suggested to improve the performances of the existing plants which can be made without disturbing whole process. These will reduce the operational cost of the plants.

For IRPs to be constructed in future it is recommended to carryout detailed water quality investigation and pilot plant /laboratory model study before construction of trealment plant in large a scale. It is also recommended to strengthen close monitoring of plant performance with special focus on day to day O&M .

CHAPTER-1
Introduction

1.1 Background:

The water sources for safe water supply in Municipal water works are surface and ground water. However, groundwater source constitutes major source of water in most of the water supply system of the country. Normally this ground water source is safe and potable in natural state, which can be supplied to the city dwellers without any treatment. But there are some areas where groundwater is contaminated by iron and other minerals. The presence of iron in water is objectionable primarily because the precipitation of iron alters the physical quality of water, impart taste and odor and turning it from a turbid yellow brown to black. In addition, the deposition of this precipitation causes staining of plumbing fixtures and laundry. Another problem which areas for presence of iron in water supply is the growth of microorganism in distribution system. Accumulation of microbial growth can lead to reduction in the pipeline diameter and clogging the meters and valves.

1.2 Present State of the Problem:

In Bangladesh 171 townships have been recognized as municipality. Till to date municipal water supply system is established in 95 towns (*DPHE,1998*). It is reported that out of these 95 towns about 60 towns experiences problem associated with iron content in the water source. The presence of iron in the water discourages city dwellers in using piped water. It results in the staining of plumbing fixtures, dishes and laundry. It also gives metallic taste and darkens tea and beverages.

For the water works, iron deposition causes encrustation in the water mains and impedes water flow. When high iron concentration are entering the system the level of encrustation sometimes becomes so high that pumping cost are increased. For the consumers iron causes partial or sometimes completely blockage of their service lines. Besides, such problems discourage new households to take connections and existing consumers to pay water taxes.

All these problems eventually affects revenue collection from the water account and the water works authority find it very difficult to keep the system in operation for want of necessary funds and other technical supports.

Several studies attempted to find the extent of area that is contaminated by iron in Bangladesh. One of such studies shows that about 65% area of Bangladesh has average iron concentration in the range of 2 to 5 ppm which is much higher than WHO guideline value for iron concentration of 0.3ppm (*Ahmed and Ahmed, 1983*). Recently DPHE in collaboration with Danida has prepared a map showing distribution of iron concentrations. This is presented in Figure 1.1 which indicates that 30% area has iron concentration with in a range of 5 to 10 ppm and 3-4% area has iron concentration higher than even 10ppm.

Under the above circumstances demand from both the consumers and water works authority to remove iron from the water is increasing day by day. The economic condition of the local water works authority is poor to take steps and invest to remove or to reduce iron content of water.

DPHE, a national agency entrusted to develop/install water supply facilities in the rural areas as well as the urban towns of the country. There are donors who are co-operating in this regard through technical and financial assistance. Among them Dutch Govt. is worthwhile to mention. The Dutch Govt. came forward to minimize the problem and installed about 5 Iron Removal Plants in different municipalities during last two-plan period. These IRPs were planned, designed and installed by the project financed by Dutch Government grant.

It has been reported from the field performance that, those IRPs are suffering from some severe problems, which affects not only their effluent quality but also operational performances. These include huge volume of backwash water requirement, surface clogging, rapid headloss, loss of filter grains etc. It is also reported that their operational efficiency is decreasing far below than the standard level. Clean water requirement has risen by 20% or more for cleaning the filter units causing added burden of O&M cost of the system on water works authority.

1.3 Rationale of the Study:

An in-depth study is required to evaluate the performances of the IRPs as to why the problems are occurring particularly within a short span of time after the plants are commissioned. Review of selection of treatment processes, hydraulic design, operational arrangement etc, is necessary to have clear

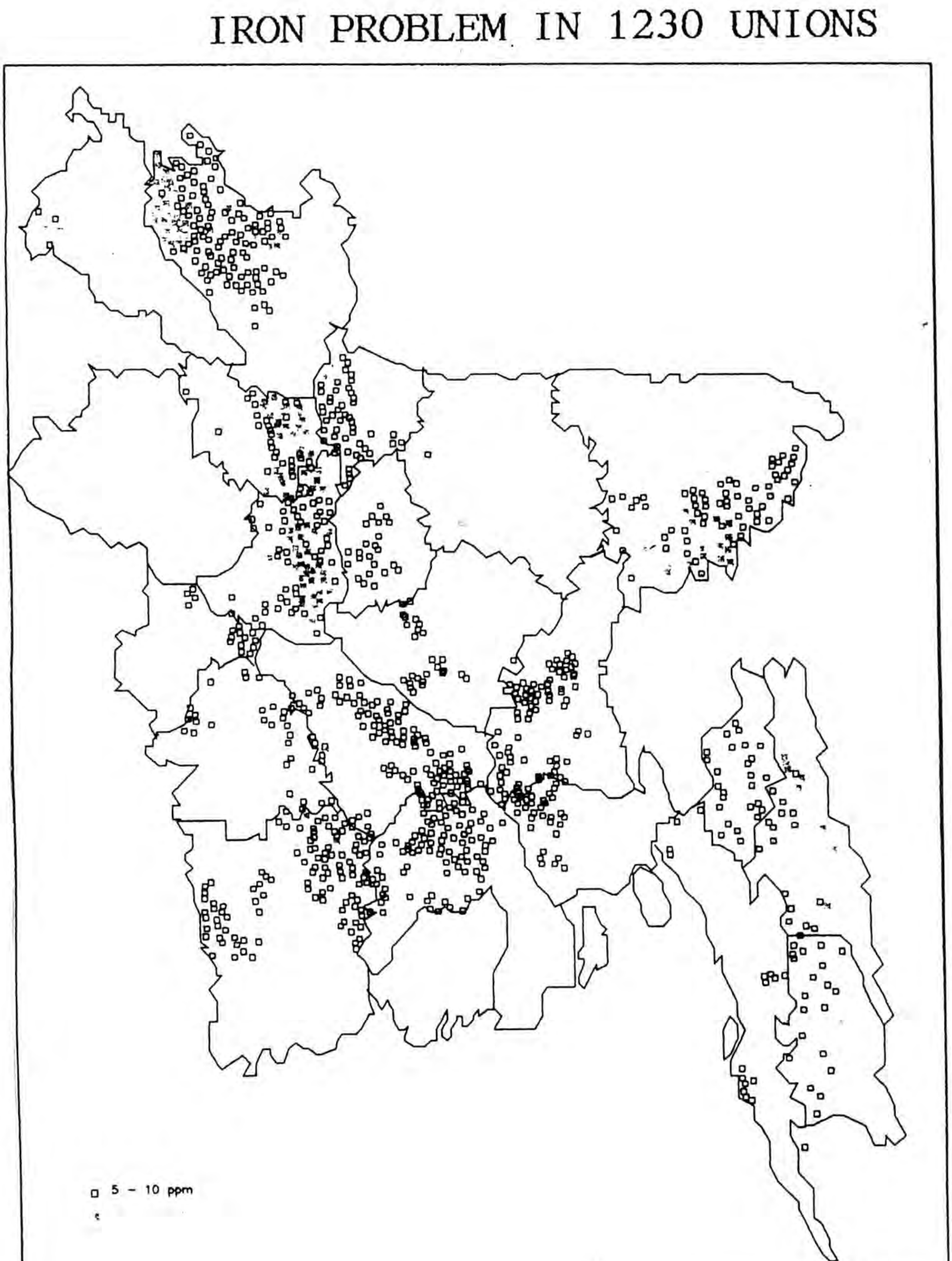


Figure 1.1: Distribution of Iron Concentrations in Different Areas of Bangladesh
(DPHE, 1996)

understanding of the problems and to find out a solution for improvement of the performance.

1.4 Selection of IRPs for Study:

For the study, three IRPs on the basis of preliminary investigation on raw water quality and operational performance etc. have been selected. The IRPs are located at the three corners of the country. It is believed that the IRPs will be representative with respect to geographic location also. It is to be mentioned here that all the plants were constructed under Dutch projects and are of similar design. The selected IRPs are: i)Dhanbandi IRP in Sirajgonj, ii)Kalibari IRP in Hobigonj and iii)Bankpara IRP in Gopalgonj district. A map showing location of the study IRP is given in Figure 1.2.

1.5 Objectives of the Study:

The Objectives of the study are as follows:

- To identify the bottlenecks of existing IRPs, constructed under DPHE-Dutch water supply program, in their whole treatment process. The possible bottlenecks include, requirement of huge quantity of clean water for back washing, reduction of filter length of run, deterioration of effluent water quality, increase of O&M cost of the plants under study.
- To review the hydraulic and process design including operational parameters in order to identify the causes of such bottlenecks.
- To propose an outline of possible modifications in the existing plant to improve their performance and also
- To propose an outline design which could result an improved performance with respect to Operation and maintenance of the IRPs.

1.6 Methodology:

Since the study intended to identify the operational & design deficiencies of existing IRPs the hydraulic and structural design parameters of the IRPs were examined. Since the IRPs under considerations are identical with respect to design & principle it was decided to select 3 IRPs for investigation. Field visits were made to investigate those physically.

Water samples were collected from influent and effluent points of the plants and tested in the BUET Laboratory for water quality investigation and to

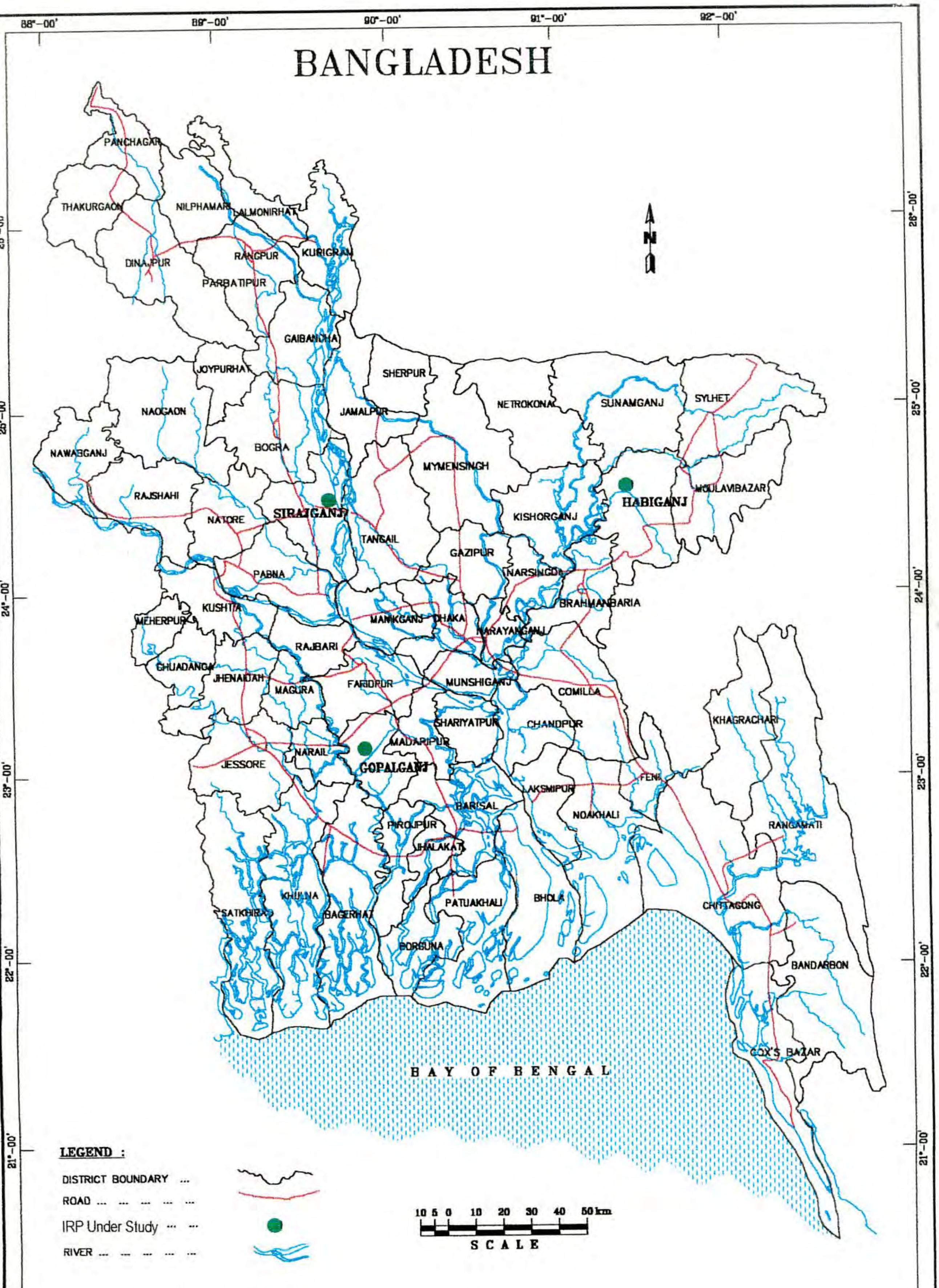


Figure 1.2 Map showing Location of IRPs Under Study

assess the selection of unit process. Water samples were also collected from the pre-filter and CWR point to examine the performance of filter units.

Some important parameters like CO_2, Fe, DO, turbidity, pH and alkalinity were measured for the raw water and for the various steps of treatment in order to determine the effectiveness of the unit process of the plants.

Back washing is one of the important phenomenons of treatment process as it governs the efficiency of filter units, which in turn affect whole treatment process. Close observations were made to assess the frequency of backwashing, quantity of clean water required for it & other associated operational difficulties. After identification of the design or operational problems, the causes were also identified. As it is mentioned earlier that, the plants are running with some operational difficulties, the cost implication was also analyzed.

A laboratory model test was carried out in the laboratory to select an appropriate unit process with an outline design of unit process. The model test will also reveal a comparative picture between the field and the standard condition.

After identifying the bottlenecks of the existing IRPs suggestions were made in order to improve the performances of the existing plants. A modified design was also proposed that would be the appropriate unit process for that particular raw water quality. It is expected that the proposed design will overcome the deficiencies of the existing plants.

1.7 Organization of the Thesis:

The first chapter of the thesis describes the background on which the study was undertaken. Objectives are then set for the study.

Chapter-2 reviews the related theories, experimental/observational results and other recommended criteria for treatment process developed time to time.

Chapter-3 provides a brief description of the existing IRPs under study. The materials and methods used in the study are described in Chapter-4.

Chapter-5 provides detailed analysis of performances of IRPs. Describes the problems that contribute to the declining performances of the IRPs.

Chapter-6 provides some design and operational modifications in order to improve performances.

Chapter-7 provides an outline design of required type of IRP on the basis of model study of raw water. Economic analysis is provided in the last chapter.

CHAPTER-2

Literature Review

2.1 Sources of Iron:

Natural Source: The presence of iron is commonly observed in water specially in ground water. It is because of the fact that Iron is one of the most abundant elements and is a natural constituent of the earth crust. The lithosphere contains approximately 5 percent iron and 0.1 percent manganese. The presence of iron in ground water is generally attributed to the solution of rocks and minerals chiefly oxides, sulfides, carbonates and silicates containing these metals. Iron occurs in the silicate minerals of igneous rocks. Pyroxenes, amphiboles and some micas, generally contain iron. It also occurs in the form of various oxides, such as magnetite(Fe_3O_4), hematite(Fe_2O_3) and limonite ($2Fe_2O_3,3H_2O$). The sulfide and carbonate minerals are also important source of iron. These include pyrites(FeS_2) and siderite ($FeCo_3$).

All the iron as described above is inorganic which refers to the clear and sparkling well waters that turn turbid on exposure to air. There is, however, other type of iron, the organic iron, which is colored with humic acid. It may present in colored well waters as well as colored surface waters.

Manmade source: Besides the natural source there are other sources of iron which includes well casing, piping, pump parts, storage tanks and other objects of cast iron and steel which may be in contact with the water and industrial wastes.

2.2 Forms of Iron in Natural Water:

Free form: Ferrous iron is known to contain either Fe^{++} or hydrated ions such as FeOH or $Fe(OH)_3$. The Fe^{2+} ion is mostly found as a hydrogen carbonate or bicarbonate in water with high alkalinity. Thus the solubility according to the laws of chemical equilibrium gives the following equations:

$$FeCO_3 \leftrightarrow Fe_2 + CO_3{}^{2-}, \frac{[Fe^{2+}][CO_3{}^{2-}]}{[HCO_3{}^{-}]} = K_{FeCO_3{}^{-}}$$

$$HCO_3 \leftrightarrow H^+ + CO_3{}^{2-}, [H^+][CO_3{}^{2-}] = K'_2$$

Substituting the later solubility equation in the former for CO_{2-}, gives

$$Fe^{2+} = \frac{K_{FeCO_3{}^{-}}}{K_2} X \frac{[H^+]}{[HCO^{2-}]} \approx \frac{[H^+]}{[HCO_3{}^{-}]} \tag{2.1}$$

Eq.2.1 shows that concentration of Fe^{2+} is inversely proportional to the bicarbonate concentration. H_2S causes the solubility drops for the low value of the product of solubility of the ferrous sulfur, leading to precipitation.

The fig 2.1 and 2.2 show that in normal pH range of natural water between 5-9, the ions in solution mainly consists of hydrated ions, Fe^{2+} and $FeOH^+$.

At pH value>12, the hydrated species $Fe(OH)_3^-$ dominates. However, the total concentration of Fe(II) determines how much $Fe(OH)_{2(s)}$ or $FeCO_{3(s)}$ will be formed at a given pH and alkalinity.

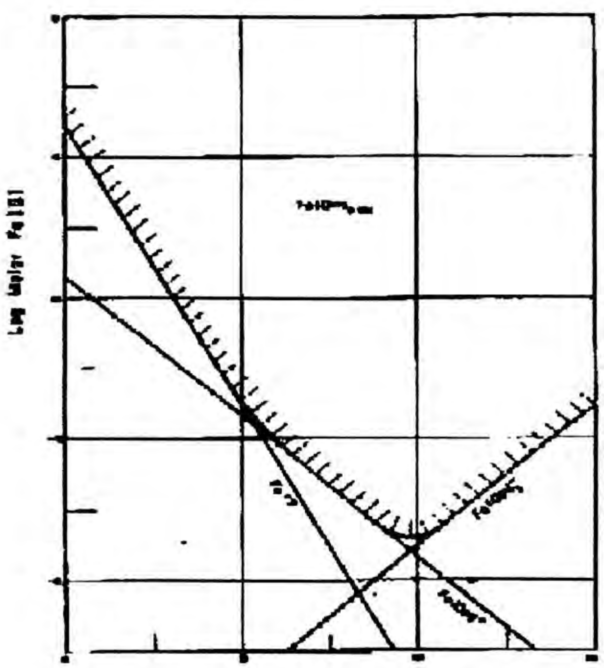


Figure 2.1 Fe(II) species in a non-carbonate non-sulphide solution (Ghosh et al, 1966)

Once oxidized the solubility of iron is severely limited over wide range of pH 4-13 by the solubility of ferric hydroxides. The Figure 2.2 shows that at pH values 6-7, aqueous ferric ions consists of Fe(OH) and Fe(OH) Complex form:

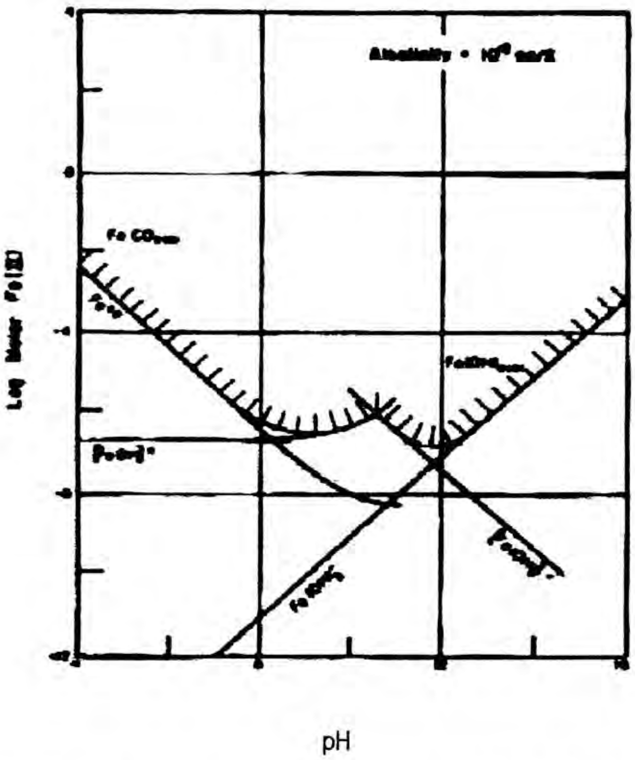


Figure 2.2 Fe(II) species in a carbonate solution (Ghosh et al, 1966)

Humic substances may form complexes and chelates with iron in water. Both ferrous (Fe^{2+}) or ferric (Fe^{3+}) iron in the form of finely dispersed inorganic and organic complexes are found. Inorganic complexes contain silicates, phosphates or polyphosphates, sulphates, cyanides etc. and organic complexes contain genuine complexation phenomena, chelation or peptization, in particular with humic, fulvic or tannic acids, etc.

Inorganic and organic complexes from both ferrous and ferric forms of iron and Fe^{2+} or $FeOH^{+}$ from ferrous iron are identified as dissolved or finely dispersed iron. Thus these do not retain on filtration. Free iron as precipitated form from both ferrous (FeS, $FeCO_3$, $Fe(OH)_2$)and ferric form ($Fe(OH)_3$ and other precipitates) do retain on the filter.

Various forms iron in water is shown on the figure 2.3

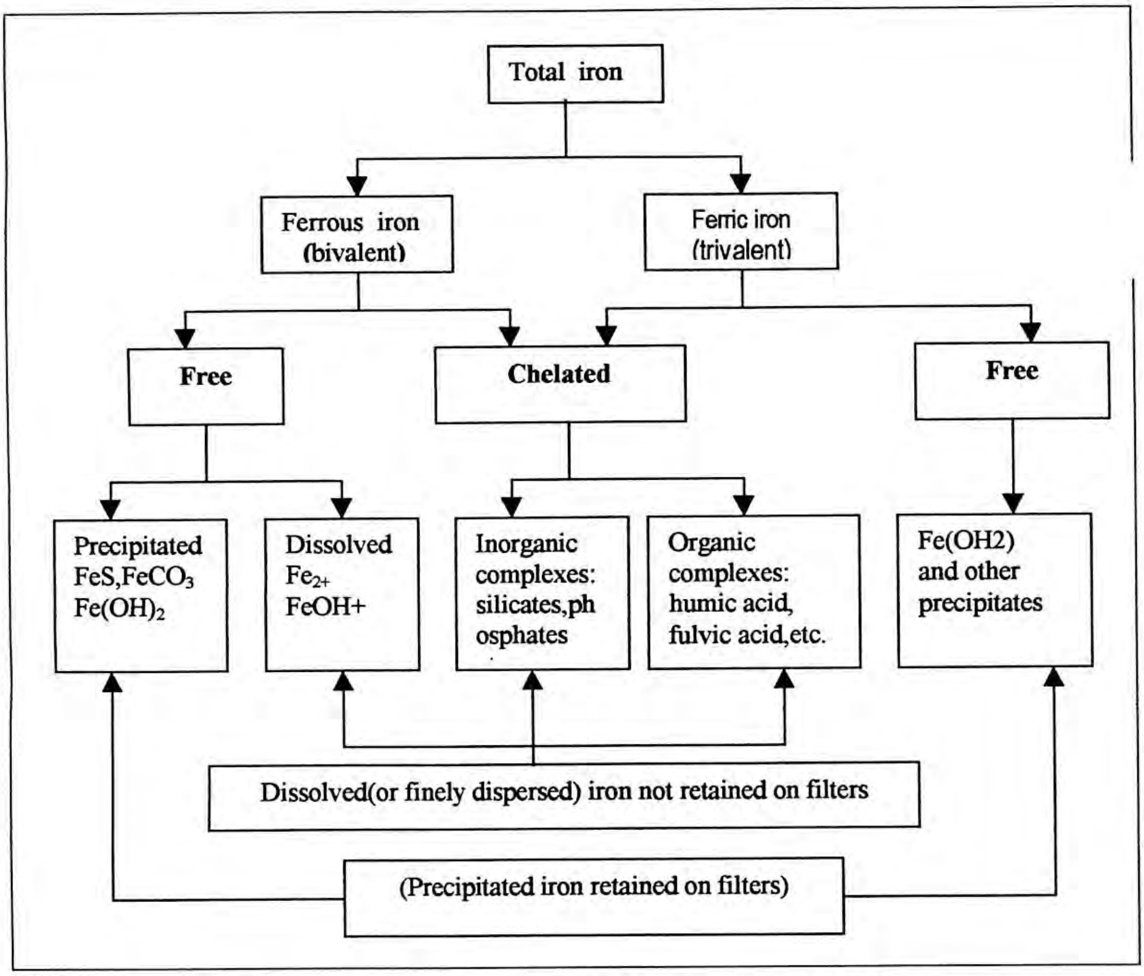


Figure: 2.3 Various Forms of Iron in Water (Degremont,1991)

The present study is intended to evaluate the performance of existing IRPs including the O&M aspect, so more detailed discussion on different complexes is not necessary. During the study, only total iron and in some cases especially for the filter influent water, filterable iron and Fe(II) will be measured.

2.3 Solubility of Iron:

As described earlier, generally both in surface and ground waters, Fe(II) and Fe(III) can be found in the form of three major solid state viz. a)$FeCO_3$(s)(siderite), b) $Fe(OH)_2$(s)(ferrous hydroxide) and $Fe(OH)_3$(s) (ferric hydroxide) and frequently FeOH and other semihydrolysed forms. The solubility of iron in natural water depend on factors like pH, alkalinity, CO_2 and concentration of organic maters.

With a reduction of a unit value of pH, the concentration of ferrous iron as $Fe(OH)_2$ can increase from 100ppm at a pH of 8 to 10,000ppm at a pH of 7. In the presence of CO_2 the solubility of ferrous carbonate governs and is 1 to 10 ppm for pH between 7 and 8 though it may be upto 100ppm for pH 6 to 7. Organic substances i.e. humic or tannic acids can create complexes with iron(II) ion holding them in the soluble state to higher pH levels. If large concentration of organic matters is present, iron can be held in solution at pH level upto 9.5.

The solution of iron bearing minerals is often attributed to the action of CO_2 in ground water, most of the CO_2 is presumably generated by the bacterial decomposition of organic matters leached from the soil. The solution of these minerals may take place under anaerobic condition and in the presence of reducing agents capable of reducing the higher oxides of iron to the ferrous state.

Waters containing high alkalinity frequently have lower iron and manganese content than water containing low alkalinity. It is also believed that water with high alkalinity have dual mechanism of iron removal i.e. removal of Fe(II) by oxygenation both by the conversion of Fe(II) to $Fe(OH)_3$ and $FeCO_3$:

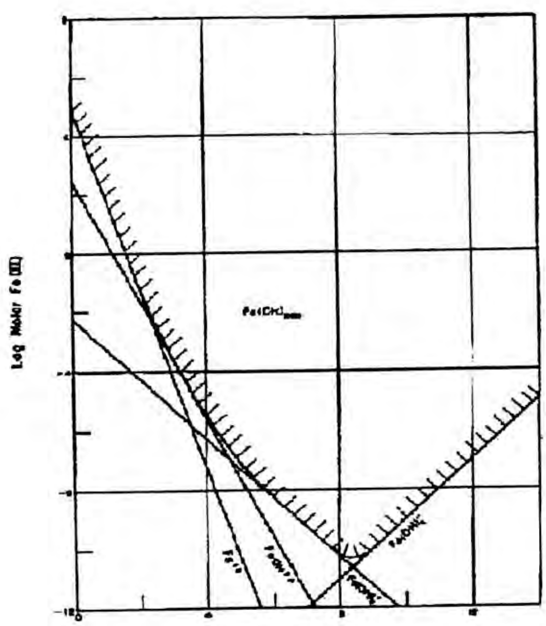


Figure 2.4: Solubility of Fe(II), Mn(II) & Fe(III) in carbonate bearing water[Ghosh et al, 1966]

$Fe^{2+} + HCO^{-3} = FeCO^{3} + H^{+}$

Inorganic complexes may be also formed where water contain substantial amount of bicarbonate, sulphate, phosphate, cyanide or halides. The formation of such complexes would tend to increase the concentration of iron found in solution. The organic complexes and chelates may also increase the solubility of iron.

2.4 Oxidation and Precipitation of Iron:

Iron can be removed from water by three methods: i) Oxidation-precipitation, ii) Ion exchange and iii) Manganese geolite process. However, the most common method as practiced in municipal water works is oxidation-precipitation method.

In oxidation-precipitation process, oxidizing agents namely oxygen, chlorine, chlorine-dioxide, hypochlorite, Manganese-dioxide, potassium-permanganate or ozone are used for the oxidation of the dissolved(reduced) iron. Subsequently, removal of the oxidized products are accomplished by precipitation and filtration.

Detention before the filtration step, provides time for the oxidation to be completed for the partial precipitation of the metallic ion by settling. Final precipitation of the iron after solubility product is exceeded, is considered to take place in the supernatant of the filter and in the pores of the filter bed.

For the precipitation of iron advantage is taken of the fact that ferric hydroxides are far less soluble than the other hydroxides and carbonates of Fe(II). Therefore Fe(II) are oxidized most commonly by oxygen, chlorine, potassium permanganate or in rare cases by ozone. Figure: 2.3 indicates that oxidation of Fe(II) to Fe(III) will greatly reduce the solubility of iron over a very broad pH range 4 to 12.

2.5 Kinetics of Oxidation

Scientists have tried to find out the rate of iron oxidation. They suggested different equations, but most of the equations are similar in nature. Gosh et al(1966) summarized the various findings as shown in table 2.2:

For homogenous oxidation the equation proposed by Stumm and Lee(1961)is widely used. In that equation, Fe^{2+} denotes the concentration of total ferrous iron, $[pO_2]$ denotes the partial pressure of oxygen, $[OH^-]$ denotes the concentration of hydroxyl ions and 'k' denotes the rate constant.

$$\frac{dFe^{2+}]}{dt} = \frac{K[Fe^{2+}][pO_2]}{[H^+]^2}$$

Table 2.1 Ferrous Iron Oxidation Rate Equation(Ghosh et. al, 1966)

Name of Scientists	Kinetic Equation
G. Just(1908)	$\frac{-d[Fe^{2+}]}{dt} = k\frac{[Fe^{2+}][pO_2]}{[H^+]^2}$
J. Hollyta (1957)	$\frac{-d[Fe^{3+}]}{dt} = k[Fe^{2+}][O_2]$
H. Boursma (1954)	$\frac{-d[Fe\ (HCO_3)_2]}{dt} = \frac{[Fe\ (HCO_3)_2][O_2]}{[CO_2\ ]}$
W.StummandG.Lee(1961)	$\frac{-d[Fe^{2+}]}{dt} = k[Fe^{2+}][pO_2][OH^-]_2$

2.5.1 Role of pH value:

The oxidation rate has been found to be strongly pH dependent. The pH dependency of ferrous oxidation rate are shown in figure 2.3 and 2.4. Fig.2.3 shows that at very low pH(<3)the rate of oxidation of iron(II) is practically independent of pH.(Stumm and Morgan,1966)

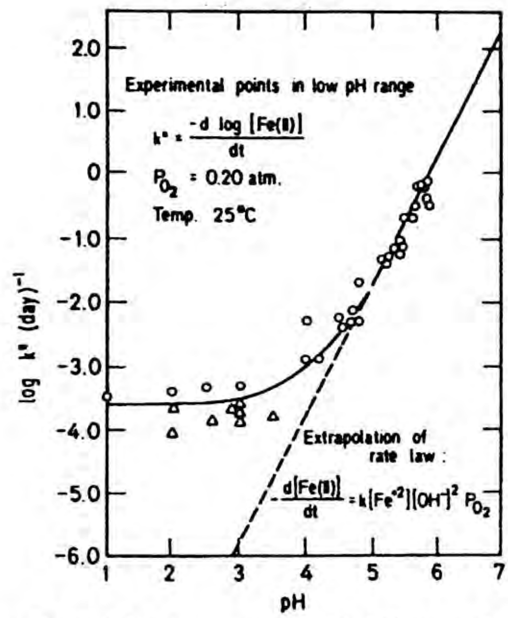


Figure 2.5 Oxidation rate of Fe(II) as a function of pH

At pH≥5 the rate of oxygenation is of second order with respect to the OH^- ions. The figure 2.4 clearly shows this dependency of oxidation of iron on pH.

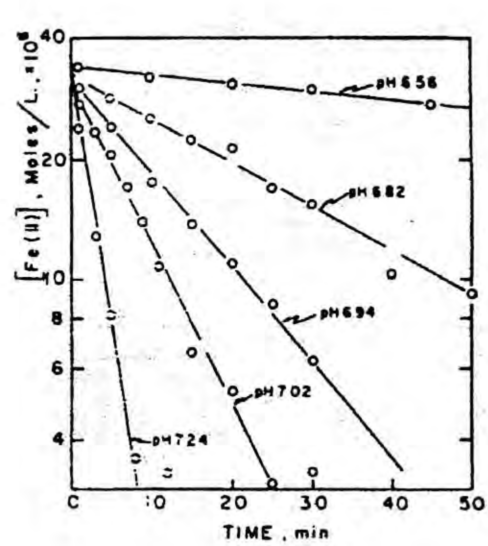


Figure 2.6 pH dependency of Fe(II) oxidation rate(Stumm and Lee, 1961)

2.5.2 Role of Temperature:

The effect of temperature on the rate of oxidation as found by Sung and Morgan(1980) is shown in figure 2.5. The reaction rate is dependent on temperature. For a given pH and ionic strength, a change in temperature 25° to 30°C causes a decrease in oxidation half time from 56 to 8 minutes i.e about 9 fold decrease. This is mainly caused by the change in OH^- concentration due to temperature dependence of the ionization constant of water.

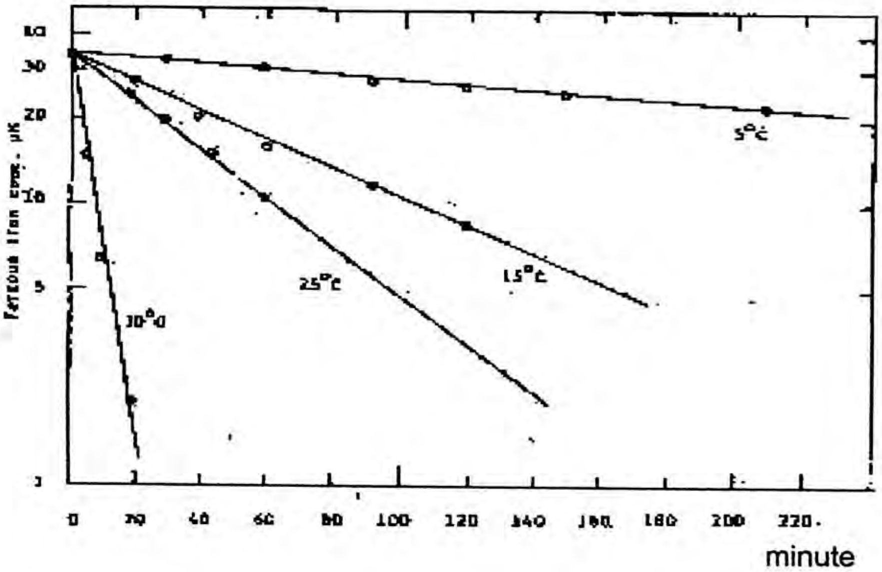


Figure 2.7 Effect of Temperature on Iron Oxidation(Sung & Morgan, 1980)

2.5.3 Role of Ionic strength:

For solutions of sufficiently low pH, the rate constant k is dependent on ionic strength (Sung and Morgan,1980). Later Sung and Forbes on further investigation observed a decrease(linear) of 'k' value upto an ionic strength of 250mM. At values greater than this, increase of ionic strength increases the rate constant.

2.5.4 Catalytic Effects:

It is also observed that a lot of substances such as Cu^{2+}, TiO_2, Silica etc. have catalytic effects on the oxidation of Fe(II) (Stumm and Lee ,1961; Barry et al 1994). It is found that for a given pH value and oxygen concentration the addition of as little as 0.02mg/l of Cu^{2+}, reduces the oxygenetion time by a factor of 5. The rate constant also depends on the nature of anion present.

2.5.5 Role of Organic matter:

Ferrous iron is capable of forming complexes with organic matter and as such, is resistant to oxidation even in the presence of dissolved oxygen. The relative strength of such complexes that has a stability constant of approximately 104.

2.5.6 Role of Alkalinity:

The concentration of iron found in solution in natural water is frequently limited by the solubility of its carbonate. Water of high alkalinity often therefore has lower iron content than the water of low alkalinity. For a given pH the solubility of iron carbonate in natural water is inversely proportional to the bicarbonate ion concentration or for most water the alkalinity.

Robinson and Dixon mentioned that in order to obtain complete oxidation of the ferrous iron, the bicarbonate alkalinity of the water should be in excess of 100 mg/l as $CaCO_3$. Generally, if the concentration of alkalinity reaches 130 mg/l as $CaCO_3$ all of the ferrous iron will be oxidized almost immediately and any further addition of chemicals would appear to be unnecessary. Low alkaline water needs some oxidizing agent ($KMnO_4$) without raising pH and alkalinity or some chemical additive to raise both pH and alkalinity.

Gosh et al observed that within a pH range of 7.49 to 7.78 an increase of alkalinity from 335 to 610 mg/l as $CaCO_3$ causes 10-fold decrease in half time.

2.6 Unit Processes of Treatment of Iron:

The unit processes of treat in conventional Iron treatment method are as follows:

- Aeration
- Coagulation
- Flocculation
- Sedimentation
- Filtration
- Disinfection

2.6.1 Aeration:

Aeration is a gas transfer phenomenon where gas molecules are exchanged between a liquid and a gas-liquid interface for an increase of the concentration of the gases in liquid phase. The concentration increases until the liquid phase is not saturated with the gas under given condition, i.e. pressure and temperature and decrease when the liquid phase is over saturated.

The gas transfer or aeration in Iron removal Process is performed by bringing air and water into intimate contact for the purpose of a) increasing the oxygen content b) reducing the carbon-dioxide content and c) removing various organic compound responsible for taste and odor.

The following factors determine the oxygen transfer rate from the air through the air water interface a) oxygen solubility in water, Cs; b) the oxygen transfer coefficient in the water, K_l; c) the available interfacial surface area and d)the temperature. The resistant against the overall oxygen transfer will be determined by the resistant in the water phase only because different coefficient of oxygen in water has significant lower value.

2.6.2 Coagulation and Flocculation:

Coagulation and Flocculation, a unit process in water treatment, requires a unique combination of chemicals and physical phenomena for producing a water acceptable for consumption. These are essential pretreatment process for the removal of finely divided particulate matter which due to its small size (usually less than 10μm) will not settle out of suspension by gravity in an economical time frame. Aggregation of fine particulate matter into larger particulate by the use of coagulation and flocculation facilities permits cost-effective removal in subsequent solid separation processes.

Formation of active coagulant species is the first step of the said process, which is accomplished through dilution, dissolution, dispersion of coagulant. Next step is to overcome the prevailing repulsive force between particulates known as destabilization. Particulate destabilization can be achieved through

a) compression of electrical double layer b) electrostatic attraction c) inter-particle bridging and d) enmeshment or 'sweep floc'. Once the destabilization is occurred accomplished contact between the destabilized particles is essential for achievement of aggregation. Less intense mixing of the particulate is provided to increase the rate of particulate encounters or collisions without breaking up or disrupting the aggregates being formed. This phenomenon is called flocculation.

Two major mechanisms flocculation are: a) Perikinetic flocculation which is the aggregation of particles as a result of Brownian motion and has the driving force in the agglomeration of destabilised particles upto 1μm.

b) Orthokinetic flocculation: For larger particulate Brownian motion is very slow and transport requires mixing by mechanical means. mechanical mixing devices such as paddles and baffles are used to provide ordered collisions due to differential particulate velocities in the mixing basin. This sub-process is termed Orthokinetic flocculation (*Kruyt, 1952*).

The important thing in this type of flocculation is the rate of flocculation and depends on the number of particles and the probability of collision. Collision may result from variable velocity of suspended particles and from micropulsation generated by mixing. The intensity of mixing can be defined by the variation in the velocity vector of fluid motion, which is described in terms of average velocity gradient. Its magnitude is a function of the useful power input, P relative to the volume, C, of the fluid and a proportionality factor, μ, the absolute viscosity. The average velocity gradient , G is thus expressed as

$$G = \sqrt{\frac{P}{\gamma C}} \quad \text{sec}^{-1}$$

Micropulsation generated during mixing not only contributes to floc formation but causes floc damage as well. Extended mixing also multiplies the recurrence of floc formation and damage, leads to the screening of active centres, decreases the flocculation rate and reduces the size of floc. CAMP has developed the flocculation criteria for optimum floc formation by combining the average velocity gradient with the mean or displacement time, which is expressed as dimentionless CAMP number, Gt. Camp analysed several flocculation basins and found satisfactory performance in basins that had the non-dimensional value, Gt, with values in the range of $2x10^4$ to $2x10^5$. Where 't' is theoretical detention time (*Fair et al, 1968*).

Ives and Bhole confirmed Camp's ideas that tappered flocculation with a declining G value was more efficient than gradient flocculation (*Ives and Bhole, 1973*).

A specific range of values is maintained for a particular condition. Thus the design and performance of a flocculator can be related to the term G x td.
In course media flocculator an expression for mean velocity gradient G is given by

$$G = 8.38\,\frac{Q}{a} X \frac{S}{d}$$

where Q=Flow rate
a= cross sectional area of bed
s= shape factor = $6/\psi$
d= diameter of coarse media

and Gtd = 3.354S/d X L(*bed depth*)

2.6.3 Filtration:

Filtration in water treatment is a major cleaning process where water is passed through a porous media. In this process the quality is made improved by part removal of suspended and colloidal matter, by reduction of the number of bacteria and other organism and by changes in its chemical constituents.

In principle, the porous media should be a stable material. In practice, bed of granular sand, anthracite, glass, cinders, crushed stone etc, are used. However, among them sand is most commonly used for its easy availability and relative low cost and satisfactory performances.(*Huisman,1986*)

Basically there are two methods of filtration:

a)*Surface filtration* which probably works by the simple mechanism of mechanical sraining.(*Cleasby,1969*)

b)*Depth filtration* by its name it is understood that the removal of impurities is occurred within the depth of the bed. Filter are also divided into two types based on flow rate namely rapid sand and slow sand filters.

Filtration Mechanism:

The mechanisms involved in removal of impurities by a filter are very complex. The overall removal is brought about by a combination of different phenomenon. Many workers have discussed the various factors which may play an important role in removal(*O'Melia and Stumm, 1967*). The dominant phenomenon depends on the physical and chemical characteristics of the suspension and the medium, the rate of filtration and the chemical characteristics of the water. However, to simplify the discussion different mechanisms are discussed separately.

Mechanical Straining: Suspended particles bigger than the pore space between sand grain are removed at the surface of the filter bed. The process is therefore independent of the filtration rate. As gradual clogging reduces pore space, the straining efficiency increases with time and form permeable layer at the surface which is known as cake filtration. This mechanism is of little significance in a filter bed composed of coarse material.

Adsorption: Removal of impurities such as small suspended particles, colloidal and dissolved substance depends on two mechanisms. First, *a transport mechanism* must bring the small particles from the bulk of the fluid within the interstices close to the surface of the media. Transport mechanisms include interception, settling, diffusion and hydrodynamic action.

Second, as the particle approaches the surface of the medium or previously deposited solids on the medium an *attachment mechanism* is required to retain the particle ($<0.01\mu m$). The attachment mechanisms may include Vandar waals forces, electrokinetic interactions, chemical bridging and surface tension.

In deep granular filter, removal results from a combination of these mechanism. Surface cake removal and depth removal may take place simultaneously. As a filter run progress the dominance of both the transport and attachment may change causing unusual patterns of effluent and head loss behavior in various filtration plants(*Cleasby, 1969*).

Length of Filter Runs

The oxidized iron together with coagulating and precipitating agents clogs its pores and increase the hydraulic loss of head. The time rate(T) at which head loss rises(H) depends on sand size filtration(d), porosity(p), filtration rate(v) and amount and character of the suspended matter in the inlet water.

$$T \,\alpha\, \frac{d^2 p^4 HSL}{v^{1.5} C_o}$$

Relationship between the factors are best determined in pilot tests. Filter run are terminated either when the head loss exceeds a reasonable value (terminal head) or when the floc breakthrough occurs. When headloss is the governing factor length of filter run normally varies inversely as the product of the initial loss of head of the clean sand bed and the square root of the filtration rate (*Fair et al, 1970*) .

Hence a compromise between head loss and effluent breakthrough is required to optimize filter runlength.

Filter Optimization:

Mintz Showed that a filter will be in an operationally optimum condition when time in which allowable limit for effluent concentration T_q and time in which allowable head loss T_h occurs i.e. $T_q=T_h$.

This can be achieved only through selection of suitable combination of grain size, filtration rate and filter depth. But the selection of suitable combination is not a easy task. It needs several trials. A combination of small grain size and high filtration rate may be attractive with respect to filter depth i.e higher Tq, but filter headloss may be so fast that may not be acceptable for good design.

Generally it is desired that head loss reached to terminal head before breakthrough occurs i.e $T_q>T_h$. In the optimization of filter, the above mentioned parameters are important however, these parameters are subject to change when influent concentrations are changed.

Filterbed Troubles:

When fine grained filtering material are used, suspended matter, iron flocs from the raw water is mostly deposited on top of the filter bed. This phenomenon is called surface cake and results a small depth 'l' of the filter bed. At the end of the filter run this thin layer are compressed by a large water pressure over it.

Mud balls: In IRPs grains carry a sticky gelatinous coating which by compression forms a tough crust. During backwashing these are broken up in smaller and larger bits. Some of the bits are so big that upward flow is unable to carry them to waste, thy remain in the filter bed indefinitely grow together again and form with adhering sand grains so called mud balls. In course of time the specific gravity of the mudballs continue to increase and find their place in the bottom of the filterbed where they grow together into mudbanks,

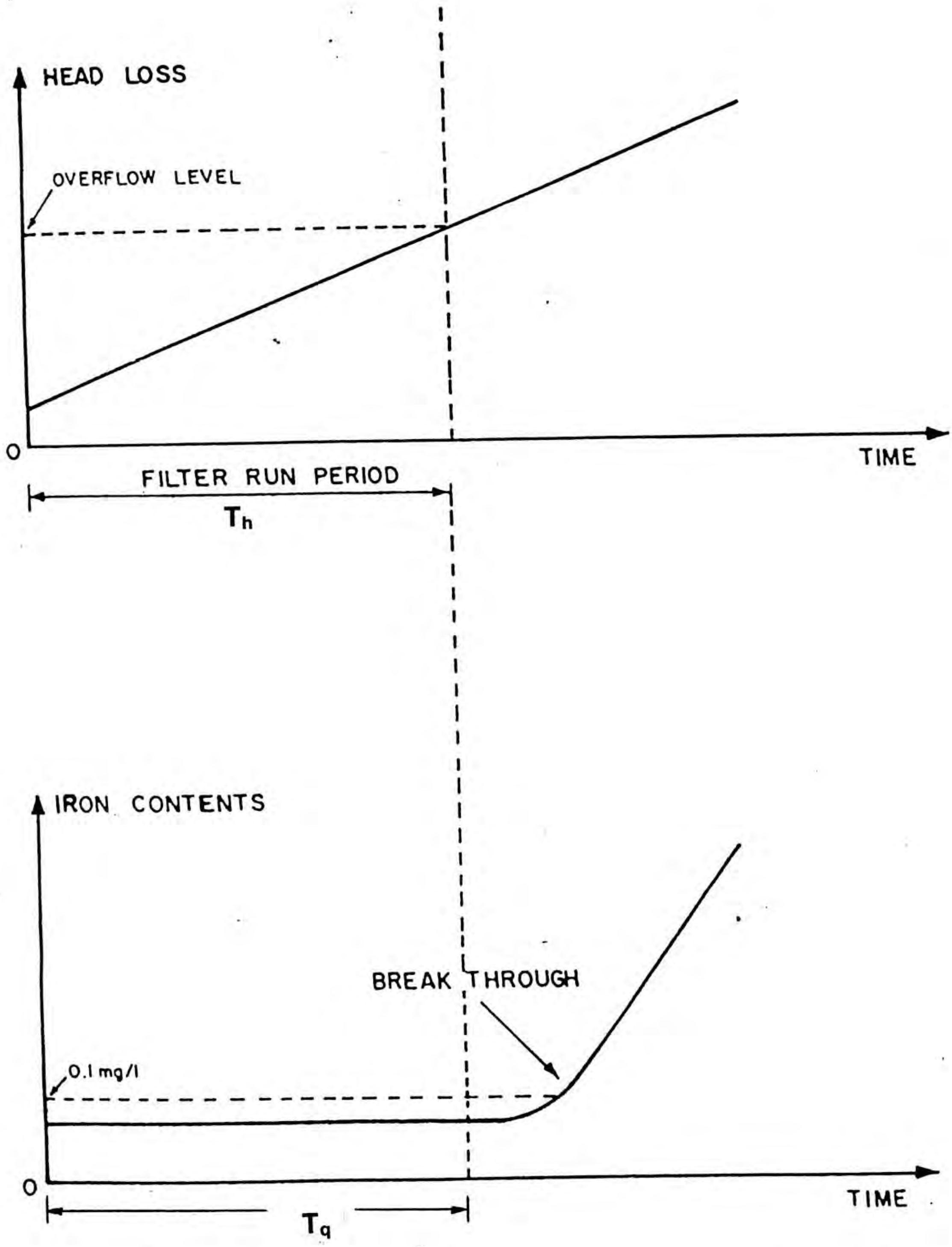


Figure 2.8: Theoretical Headloss and Filter Effluent Quality Curve

clogging part of the filter bottom which cause: a) increase of headloss b) shortening filter run.

High velocity backwash has been used in the USA to attemp to remove this problem but it creates appreciable loss of filtering materials(*Cleasby, 1971*).

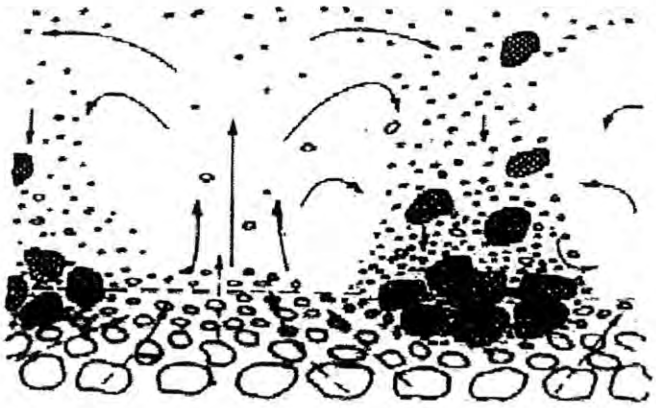


Figure 2.9: Formation of Mudballs in filter bed

Filter cracks: Along the wall of a filter box, the resistance against downward water movement will always be smaller than in the filter proper. Head losses along these walls will consequently be less than in the body of the filterbed, giving rise to an excess water pressure, which tries to move the filtering materials away from the walls. With clean coarse grained sand this will have no adverse effects but with fine grained materials filter cracks may develop when by surface filtration the pressure difference are larger and the grains are coated with soft and compressible materials. Through these cracks raw water may penetrate the filter bed to great depth reducing filtration efficiency and deterioration effluent quality the deposition of suspended matter from the raw

Figure 2.10: Formation of Filter Crack

water in these crack, will also result in mud banks extending now from the walls into the filtered. And again disturbing both the process of filtration ad backwashing.

The filterbed troubles of mudballs and filter crack are primarily due to the use of fine-grained filtering material. The best way to avoid these filterbed troubles is the use of coarser filtering materials, which on one hand can be kept cleaner by backwashing water alone, and allowing on the other hand a deeper penetration of suspended and colloidal matter from the raw water(*Huisman,1986*).

If the filtration rate on a filter which contains deposited solids is suddenly increased, the hydraulic shearing forces also suddenly increase(*Cleasby, Tuepker, 1970*). This disturbs the equilibrium existing between the deposited solids and the water, and some solids will be dislodged to pass out with the effluent. Depending on the type of solid and the magnitude and suddenness of the rate change, the effect can be quite drastic. All sources of the sudden rate change should be avoided in design.

Underdrainage:

The main purposes of the underdrainage system are to support the filter medium, collects the filtered water, distribute the backwash water evenly over whole area of filter bed and prevent loss of the filter medium with the filtered water. Some common types are described below:

1. Manifold and laterals: This system one of the oldest and still most widely used filter bottom, consisting of a manifold to which a series of laterals are connected as shown in Figure 2.10(A). Laterals are provided with openings or orifice in the lower portion. Through these opening the washwater directed downward either vertically or under an angle of 30^0 to 45^0 with the vertical. In both cases the kinetic energy is of the jet emerging from the opening is dissipated by collision with the bottom of the filterbox or the sides of the surrounding pebbles and there is no danger of disturbing the filterbed. The perforations vary from 6.5 to 12mm in diameter. 0.5 to 0.7m gravel graded from 3 to 55 mm is generally provided.

The merits of this system over others are i) cheaper ii) can be constructed locally, no involvement of Foreign exchange iii) excellent performance iv) when feel so, that washing with water alone is no longer sufficient, air scour system can be introduced without major change and v) long economic life.

2. Wheeler filter bottom also shown in figure 2.10(C) consist of a false concrete bottom supported above the filter box by piers. Uniformly spaced, inverted pyramidal depressions contain several specially proportioned porcelain spheres and one porcelain orifice at the bottom through which the water flows. Here a 300mm layer of gravel graded from 25 to 3mm is commony provided.

3. Perforated precast underdrain system are commonly self- supporting and completely cover the bottom of the filter box. They form the channels to collect and carry the filtrate to a central conduit during filtration and to distribute the washwater from the same conduit during backwashing. One common type, the Leopold bottom shown in figure 2.10(B) is made of vitrified tile and has many 4 to 8mm orifice closely spaced on its surface. The smaller orifices reduce the depth and upper size of gravel required.

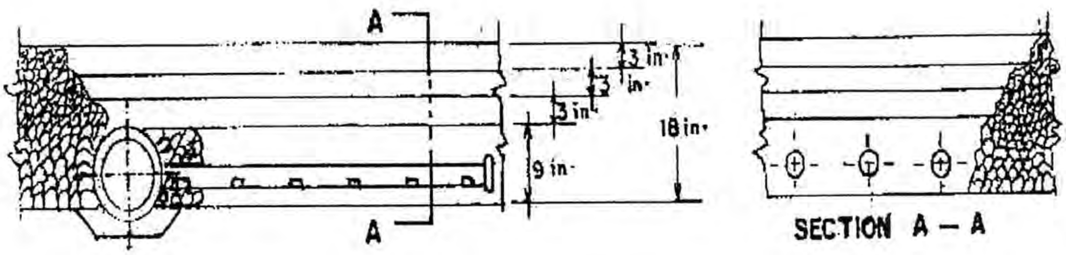
4. Filter bottom with nozzle: It is made for separate or simultaneous air–wash by providing the strainer with long stem anchored into false bottom. The principal function of the nozzles is to distribute wash water with out a jet action. Nozzles also may not be suitable for use in filtering water that is lime softened or carries high iron or manganese content because of clogging problem (*Huisman, Schippers, 1996*). In this system no supporting gravel layer is required.

2.7 Backwashing:

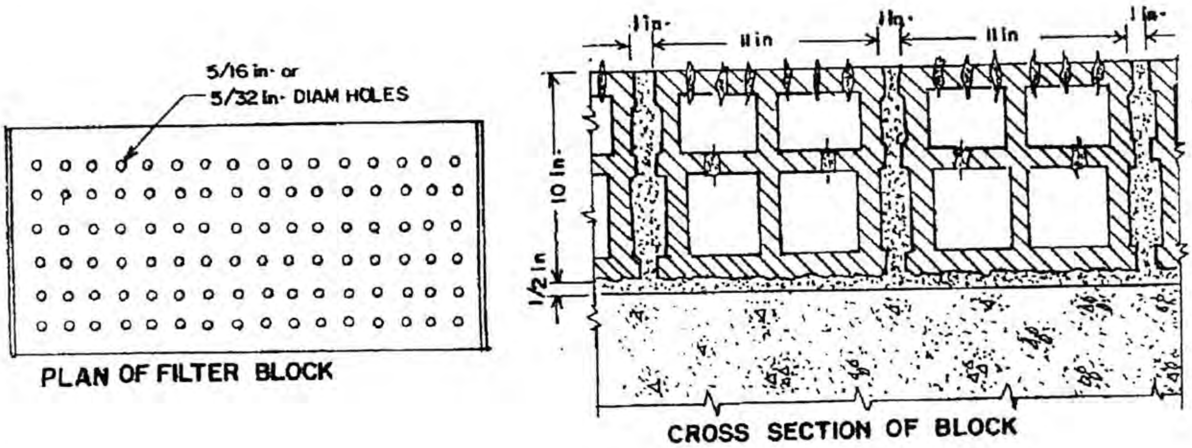
During the filtration hydraulic resistance occurs due to clogging and when it reaches the maximum value (commonly, 2.5-2.8m) or the quality of the effluent drops below the predetermined set standards, the functioning of the filter is stopped. To restore the purification capacity of the bed, reverse flow of filtrate used to wash out the accumulated deposits. Back washing accomplishes two purposes (i) scouring or dislodging of impurities attached to the filter grain surface by shearing action of air and water. (ii) expansion of filter bed to increase the pore space with a view to easy escape of the impurities with the wash water.

2.7.1 Drainage of Backwash water:

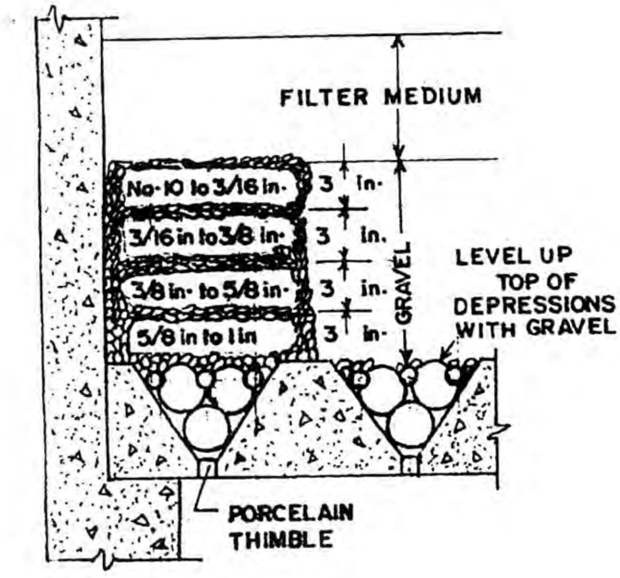
The washwater together with the impurities removed from the opening between the sand grains must be disposed off through a system commonly known as trough or gutter. The upper overflow edge of the washwater trough should be placed sufficiently near to the surface of the sand so that the washed out impurities are removed easily and in short time and no large



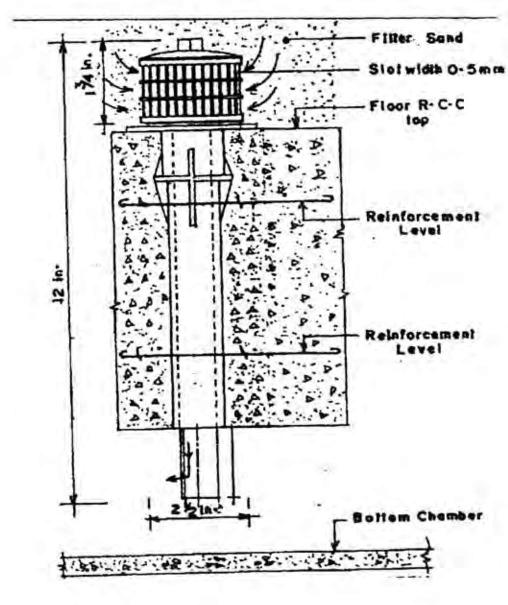
a)



b)



c)



d)

Figure 2.11: Filter Underdrain System ; a) Lateral-Manifold b) Perforated Pre-cast Floor c) Wheeler Filter Bottom and d) Filter Strainer-Nozzle type

quantity of washwater is left in the filter after completion of washing. On the other hand, however, this upper edge should be set a minimum distance of about 0.25m above top of the expanded sand bed to prevent loss of sand during washing as much as possible. For the same reason the trough must be kept at lest 0.05m above the expanded sand bed. The cross sectional area of the trough should be large enough to carry the maximum amount of wash water with at least 0.05m freeboard. The depth h_2 at the outlet end of the trough depends on the conditions prevailing in the central gutter. The depth h_1 at the other end can be calculated with momentum theory. For rectangular cross section constant flow Q m^3/sec. *(Fair, 1970.)*

$$h_2 = \sqrt[3]{\frac{Q^2}{gb^2}} \qquad h_1 = \sqrt{3}h_2$$

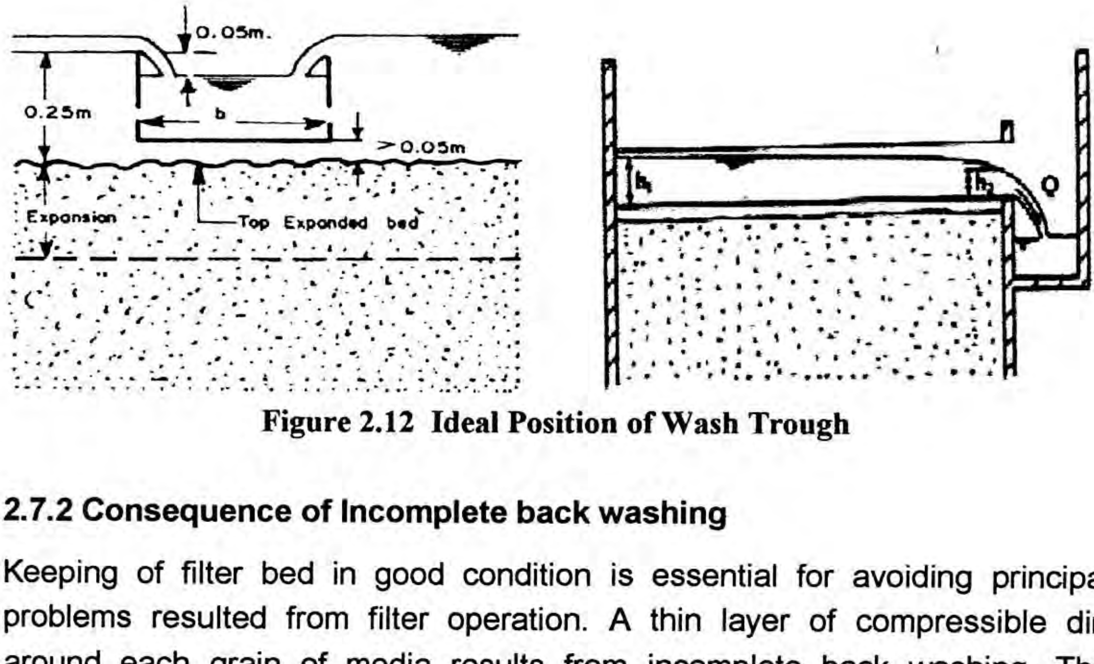


Figure 2.12 Ideal Position of Wash Trough

2.7.2 Consequence of Incomplete back washing

Keeping of filter bed in good condition is essential for avoiding principal problems resulted from filter operation. A thin layer of compressible dirt around each grain of media results from incomplete back washing. The deposits of solids near the surface of the media leads to the formation of muddballs gives cake filtration causing rapid headloss development.

2.7.3 Frequency of back washing and consumption of wash water

Frequency of backwashing and consumption of wash water depends on the nature of the water to be filtered. In practice, loss of head is taken as a criterion. Washing is done when it reaches a certain limit (terminal head) or when filter break-through occurs (*Baylis et al, 1971*).

Consumption of wash water is related to the character and weight of the particles retained per m^3 of filtering material and applied method of washing. For example, the combined use of air scour and water requires less by some 20 to 30% water as compared with washing with water alone. The following factors increase the wash water consumption:

- The deeper the water above sand and the smaller the backwash flow rate.
- The greater the distance between the wash water gutter and the greater the volume of sludge to be removed.
- The greater the cohesion and density of the sludge.

According to Degremont's experience, wash water consumption for a one meter deep bed washing with water and air, and washed with water only requires the consumption of 3 to 40 m^3 /m^2 respectively. Wash water uses in direct filtration is greater than for conventional treatment as high as 6% as compared with 3 to 4% for conventional plant due to shorter filter runs.

2.7.4 Criteria for water alone backwashing

Amirtharajah and Cleasby described that, the cleaning of granular filters by water backwash alone to fluidized bed and this abrasion between the filter grains is negligible. The hydrodynamic shear at the water filter grain interfaces accomplishes cleaning in a water fluidized bed. i.e., the principal mode of cleaning by hydrodynamic shear (*Amirtheraja, 1978*).

Amirtharajah shows theoretically and experimentally that maximum hydrodynamic shear occurred in a fluidized bed at expanded parasites of about 0.70 to 0.72 for typical sand which corresponds to about 25-30% expansions in the top layers where most of the particles are deposited during filtration.

However, Jhonson and Cleasby showed from experimental observation and theoritical considerations that less expansion is needed with coarser sand. 18 to 20% expansion was taken as the optimum value for the said sand.

Due to the inherent weakness of water backwash as mentioned above for its less abrasion capacity. Thus, it requires auxiliaries such as surface wash or air scour, which provides, required collisions in the media for effective cleaning. Surface wash is essentiality limited to cause collisions at the top layer of the bed whereas, the air scour causes collision through the bed.

It could be mentioned that, air scour followed by water backwash is a common method of cleaning filters in European countries, and it has been in use for over 50 years. Backwashing with water alone is the common U.S. practice. This system is associated with some filter bed troubles which are not common where the air and water wash system are being used.

The author of this thesis while visited 'Mumbai Water Works' observed that filters were being backwashed with air scour for 3 minutes followed by water wash for another 6 minutes. Total requirement of backwash was about 4% of the water production. It was also observed that even and homogenous cleaning of the filterbed was also ensured through cross cleaning. Cross cleaning is a process of jetting raw water along the two wall of the filter run parallel to the trough.

2.7.5 Air scour in Backwashing

It applies the distribution of air over the entire filter area at the bottom of the filter media and is used in several ways to improve the effectiveness of backwashing and or obtain the use of lower backwash water flow rates. The use of air prior to the water backwash or concurrently with water backwash is possible. Concurrent use has the concern over loss of media to the overflow when water reaches the overflow level for violent agitation resulted from air scour. *In order to prevent loss of filter media during the air scour, using air only, the water level is to be lowered 6-8 inch below the overflow level.*

For dual media hydraulic backwash at a rate to fluidize the bed and restratify the media is generally employed. Lower rates of backwashing (subfluidizing) can be applied for mono media filters. When they are subject to supplemental scouring i.e. air scour.

2.7.6 Backwashing Method and Media Size

The filter media effectively controls the performance of a filter. Theoretical and empirical models and experimental studies have indicated that optimum designed are obtained using coarse uniform single media (>1mm) and extra deep beds (>4ft), with air scour for backwash to maintain destratification (*Ives and Hang, 1969*). This design approximate European practice (Baylis et al,1971). Media design is frequently controlled by backwashing requirements. According to Degremont different grain size for filter bed have different washing requirements, among them a few can be mentioned.

Effective size from 0.60 to 0.8mm

This can be applied without prior clarification, with or without coagulation on the filter (turbidity less then 50 mg/l of silica) with a filtration rate of 7 m/h. Washing of this kind of filtering material by water alone, and wash water and air scour is possible.

Effective size from 0.90 to 1.2mm

Continental Europe uses this standard grain size in a homogeneous layer to filter clarified water or water with a low turbidity with coagulation on filter. It is also used for direct filtration of water and is ideally suited for false floor filters washed water and air filtration rate of upto 15 and 20 m/h.

2.8 Recent development in filtration, Direct filtration.

2.8.1 Introduction

Direct filtration has received attention in the treatment of drinking water in 1970s. Recently, direct filtration has been developed in Europe, Canada, United States and Germany. This treatment includes a treatment scheme where all of particulate are removed in the filters, there is no sedimentation or floatation prior to filtration. Two methods can be identified: (a) direct filtration without a prior separate flocculation unit. The process refrred to as *in-line filtration* or sometimes called *contact filtration* with flocculation occurring in the filter itself. Destabilization of suspended solids occurs immediately before filtration at rapid mixing unit with coagulant addition. Only micro flocs are allowed to form. This system has much higher rate of orthokinetic flocculation within filter bed than conventional system. b) The second method includes the provision of separate flocculation step prior to direct filtration and to be distinguished as flocculation with a view to formation of larger floc.

According to Degremont, a third method similar to others except without using any chemicals. He described that direct filtration of a liquid where the suspended solids retain their original state and electrical charge will therefore be very different from filtration of a coagulated liquid.

2.8.2 Mechanism

Water containing the destabilized particles can be taken directly to granular bed in which contact flocculation takes place as part of the filtration process. Surface of the filter grains absorb the floc particles, and this absorption or contact increases with smaller filter grains for more rapid removal if particulate matter.

According to several investigators the mechanism of suspended particle removal within filter bed of direct filtration is likely that of rapid filtration. Wilson et al. in their study, described that, direct filtration requires very small strong floc, unlike the larger flocs necessary for sedimentation. Strong flocs can penetrate the pores of the media and withstand the high shear regime. A number of researchers including Ives state that once the particle attach itself to a filter grain, it does not become detached if the approach velocity is kept constant.

2.8.3 Raw Water Quality for Direct Filtration

Water quality varies in places, in seasons and in sources. Thus, selection of treatment method should be made on particular situation. Nevertheless, a number of efforts to define acceptable source water for direct filtration have been made, most of them were based on pilot or full-scale observations. The values from them provided a preliminary indication only, reliable value would be obtained from pilot plant tests considering the prevailing raw water quality of the place.

An American Water Works Association(AWWA) committee report defined water meeting the following criteria as a perfect candidate for direct filtration (*Committee Report*).

Color	< 40 TCU
Turbidity	< 5 NTU
Algae	< 2000 asu/ml
Iron	< 0.3 ppm
Manganese	< 0.05 ppm

Cleasby et al. Considered the AWWA committee guidelines acceptable except for turbidity which they consider too low. During low algae season they suggested turbidity limits of 12 NTU.

Despite the different values it is possible to summarize them in a comprehensive consideration as follows:

-	Turbidity	$\leq$ 10 units
-	Particle volume concentration	$\leq$ 2ppm
-	Particle mass concentration	$\leq$ 10mg
-	Particle size	$\leq$ 20μm
-	Coagulant dosage	< 6-7mg/1Fe or Alum
-	Diatoms	$\leq$ 1000 asu/L
-	Coliform MPN	< 90/100ml
-	TOC	< 5 mg/1
-	Color	< 15 unit

CHAPTER-3

Brief Description of IRPs

3.1 Location:

Sirajgonj: The first IRP under this study is located in Sirajgonj. Sirajgonj is a upgraded district town and located at distance of 130 km northwest from Dhaka. The IRP has a design capacity of $204m^3$/hr. and was constructed at Dhanbandi, Sirajgonj town under the Netherland-Bangladesh cooperation program in 1991.

The water resource development master plan of Sirajgonj indicated ground water as a promising source for water supply with respect to availability and cost effectiveness. However, the ground water contains high concentration of iron which causes aesthetic problem and dissatisfaction among the community. This led to the construction of IRP in Sirajgonj.

Hobigonj : The next IRP under the study is in Hobigonj. It is also a upgraded district town and is located on the west bank of river khowai at a distance of 166km northeast from Dhaka. The IRP has similar capacity as that of Sirajgonj ($204m^3$/hr) and was constructed at Kalibari, Hobigonj town under Netherland-Bangladesh cooperation program in 1992.

Like in case of Sirajgonj choice for source went to ground water on the ground of source availability and cost effectiveness. The water quality analysis report for Hobigonj revealed that the ground water contains iron beyond acceptable limit as described in WHO guideline for drinking water quality. The iron content is in the range of 3.5 to 7.00 ppm. The master plan for Hobigonj water supply indicated the nessecity for treatment of ground water to make it potable for public use.

Gopalgonj : The IRP in Gopalgonj is located at Bankpara, Gopalgonj town. The IRP has similar capacity as that of sirajgonj ($204m^3$/hr) and was constructed under Netherland-Bangladesh cooperation program in 1992.

The said cooperation program selected ground water as the source for public water supply system despite the fact that groundwater quality is very poor with regard to mineral content. The iron content varies from 5 to 15ppm, the chloride content varies from about 50 to over 500 ppm. The program has constructed one IRP which can remove iron only without taking care of chloride content.

It is worthwhile to mention here that Gopalgonj town has an alternative source of water, surface water. The old Modhumoti river is flowing beside the town. This water source appears to be most potential with respect to access, quantity and quality to some extent.

3.2 Unit processes of the existing Iron Removal Plants

The basic processes in iron removal are

Aeration to convert the soluble ferrous compound to insoluble iron hydroxide flocs. Aeration is being done by cascades in the plants.

Filtration of aerated water to remove formed iron hydroxide flocs in a sand filter bed. In all the plants under study rapid sand filters are used for this purpose.

Storage: Underground reservoir is constructed to retain water in order to facilitate distribution and cleaning of the filters.

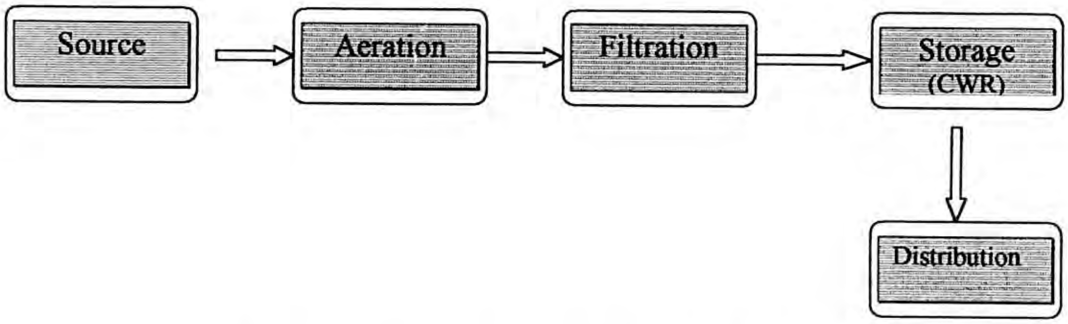
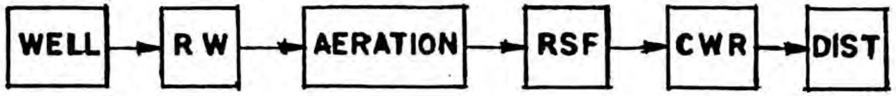


Figure 3.1: Flow Chart of Existing IRPs

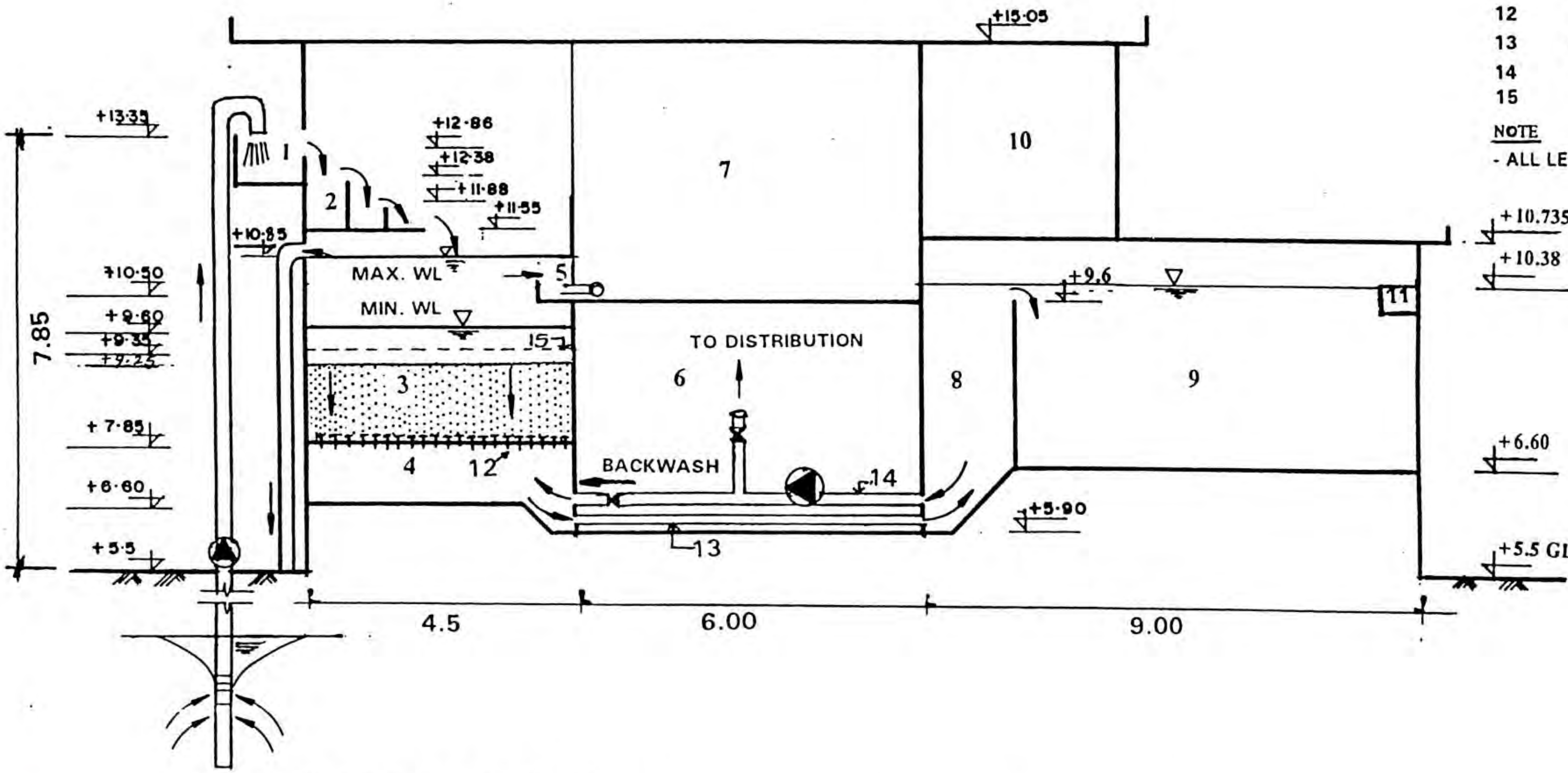
A sketch map of the IRPs under study is given in Figure 3.2 to 3.4. The detailed dimensions and design data of the different units are given in table 3.2 to 3.3.

: PROCESS SCHEME IRP SIRAJGANJ

PROCESS SCHEME



SKETCH PLANT



WELLS, FILTERS(+ CASCADE),
2 NOS EACH 13.5m²

LEGEND

1 RECEVING WELL[5 M3]
2 CASCADE
3 FILTER SAND
4 UNDER DRAINAGE CHAMBER
5 BACKWASH GUTTER[0.5 m sqr]
6 PUMP ROOM[39.30 m^2]
7 OPERATING ROOM
8 EFFLUENT WEIR
9 CLEAR WATER RESERVOIR[194 m^3]
10 CHLORINATION ROOM
11 OVERFLOW CHANNEL
12 FILTER NOZZLE [50 nos/m^2]
13 FILTER EFFLUENT PIPE[Φ 200]
14 BACKWASH/DIST. PIPE[Φ 250]
15 DESIGN SAND LEVEL

NOTE

- ALL LEVELS & DIMENSIONS IN METER.

Figure 3.2 Process Scheme of Existing IRPs Sirajgonj

PROCESS SCHEME

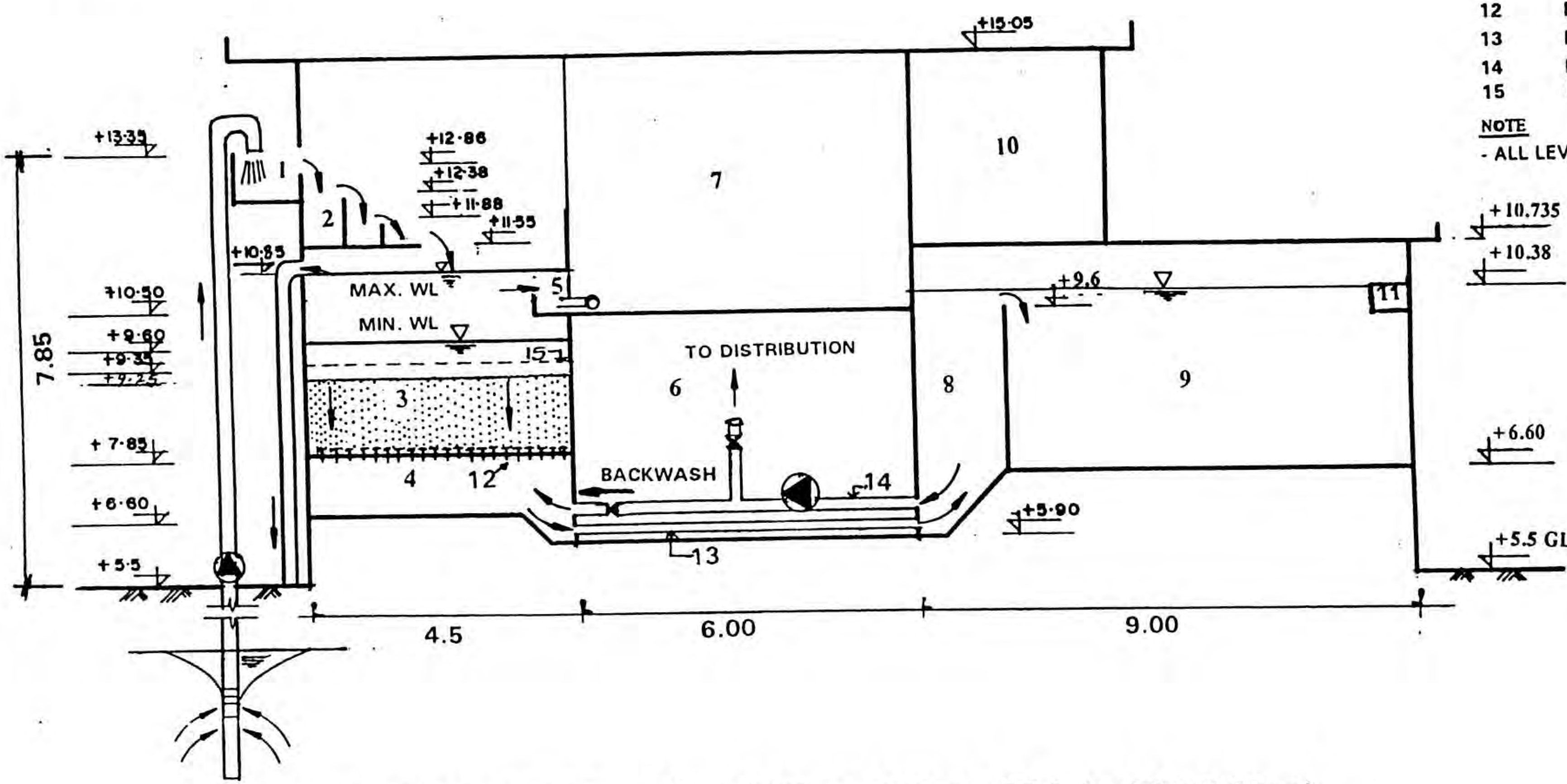
WELL → R W → AERATION → RSF → CWR → DIST

LEGEND

1 RECEVING WELL(5 M3)
2 CASCADE
3 FILTER SAND
4 UNDER DRAINAGE CHAMBER
5 BACKWASH GUTTER(0.5 m sqr)
6 PUMP ROOM(39.30 m^2)
7 OPERATING ROOM
8 EFFLUENT WEIR
9 CLEAR WATER RESERVOIR(194 m^3)
10 CHLORINATION ROOM
11 OVERFLOW CHANNEL
12 FILTER NOZZLE (50 nos/m^2)
13 FILTER EFFLUENT PIPE(Φ 200)
14 BACKWASH/DIST. PIPE(Φ 250)
15 DESIGN SAND LEVEL

NOTE
- ALL LEVELS & DIMENSIONS IN METER.

SKETCH PLANT



WELLS, FILTERS(+ CASCADE),
2 NOS EACH 13.5m^2

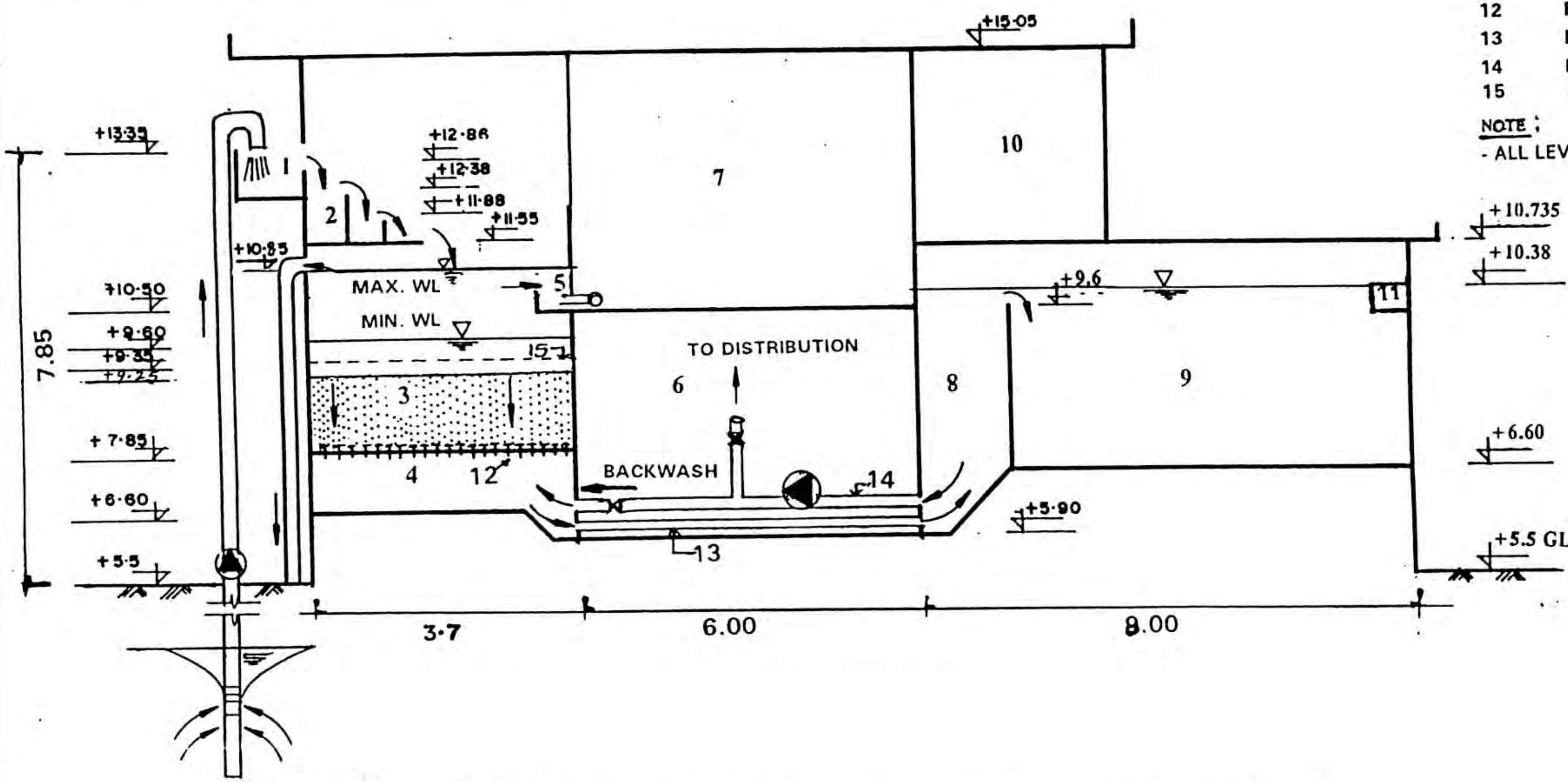
Figure 3.3 Process Scheme of Existing IRPs Hobigonj

PROCESS SCHEME IRP GOPALGANJ

PROCESS SCHEME

WELL → R W → AERATION → RSF → CWR → DIST

SKETCH PLANT



LEGEND

1	RECEVING WELL[5 M3]
2	CASCADE
3	FILTER SAND
4	UNDER DRAINAGE CHAMBER
5	BACKWASH GUTTER[0.5 m sqr]
6	PUMP ROOM[39.30 m²]
7	OPERATING ROOM
8	EFFLUENT WEIR
9	CLEAR WATER RESERVOIR[100 m³]
10	CHLORINATION ROOM
11	OVERFLOW CHANNEL
12	FILTER NOZZLE [50 nos/m²]
13	FILTER EFFLUENT PIPE[Φ 200]
14	BACKWASH/DIST. PIPE[Φ 250]
15	DESIGN SAND LEVEL

NOTE :

- ALL LEVELS & DIMENSIONS IN METER.

WELLS, FILTERS(+ CASCADE),
2 NOS EACH 93 m²

Figure 3.4 Process Scheme of Existing IRPs Gopalgonj

Nevertheless, a brief description of operation at different steps of the plants are given in the following sections to understand the working process easily:

3.2.1 Source:

Raw water is supplied to the IRPs directly from a number of production tubewells. At present none of the plants are receiving designed quantity of water from the wells. Table 2.1 reveals that only 72 to 78% of design flow is observed during the study. The operating hour of the wells are equal to the operating hour of the IRPs.

Table 3.1 Design and Actual flow into the IRPs

	Sirajgonj		Hobigonj		Gopalgonj	
	Design	Actual *	Design	Actual	Design	Actual
Water flow (m3/hr) % of Design flow	204	160 (78%)	204	157.26 (77%)	140	100.4 (72%)

** data for 1998*

3.2.2 Water Intake:

The water intake in the IRPs are the division chambers. Raw water from the production wells enters the division chamber where it is equally divided over two filter units by means of V-notch. The water flow can be measured with the help of these V-notch. For this purpose measuring scales are mounted to measure the height of the water level. The water discharge can be computed with the help of a conversion table.

3.2.3 Aerator:

From the division chamber, the raw water flows via the V-notches to the cascade aerator. For iron removal it is necessary to aerate the anaerobic raw water. The atmospheric oxygen brought into the water reacts above pH=6.5 quickly with the dissolved ferrous compound converting them into insoluble ferric hydroxide.

The reaction is $4\ Fe^{2+} + O_2 + 10\ H_2O = 4\ Fe(OH)_3 + 8H^+$

This ferric hydroxide is removed by filtration. During aeration carbon dioxide is simultaneously removed from the water increasing the pH, which in its turn increases the conversion rate of ferrous to ferric hydroxide.

3.2.4 Rapid Sand Filters:

After the last step of the cascade the aerated water fall into the rapid filters. Rapid filtration is a purification process whereby the water to be treated is passed through a porous medium at relatively high velocities, about 7.5 m/hr. The filterbed consists of a 1.5m thick sand layer. The grain size of the filter sand is 1.2 – 1.5mm.

The rapid sand filter is consists of following components:

- One filter bed and concrete chamber.
- Filter media
- An artificial filter bed where suffiecient nozzles are fitted to flow a defenite quantity of water to wash the medium.
- Water inlet, outlet for filtered water and valves.
- Wash water trough.

During the process of filtration the suspended solids are removed from the water and accumulated on the sand grains and in the pores between the grain. Due to clogging of the pores the resistance of the filter bed increases and consequently the water level on the top of the filter rises. This water level is limited by an overflow pipe.

When the filter run period is approached, the iron concentration in the filtered water can be slightly increased with time at a certain moment. However, a steep increase in turbidity due to iron concentration occurs rather suddenly which is called the break through of the filter. The suspended solids cannot be retained adequately any more by the filter bed so that the bed must be cleaned or regenerated.

The sand filter are designed in such a way that overflowing (maximum allowable water level) precedes the "break-through" of the filter bed. When overflowing is observed the filter must be cleaned to remove the accumulated suspended solids to restore the original water level over the filter bed. This cleaning is carried out by back washing the filter with clean water.
The time between two successive backwashes of a filter bed is called the filter run length.

Back-washing: Cleaning of the filters is performed by reversing the water flow through the filter. This expels and carry the accumulated suspended solids to waste. The water of the CWR is used for backwashing of the filters. After performing its duty in cleaning the rapid filter bed, the washwater is discharged via the backwash gutter and drain to the concrete made sludge tank.

The backwashing is the main aspect of the operation and maintenance of the filter and requires the operation of several valves. It is initiated on basis of high water level by the operator.

Filterbed materials: The filterbed material is of utmost importance for the filtration process. Sand is suitable filter medium because it is clean, durable and widely available at low cost. But as found in nature the variation in grain size of the sand is too large that fine material gives a short filter run length due to rapid clogging while coarse material does not add to effluent quality. Therefore the natural sand must be sieved to remove coarse and fine fraction. In the IRPs the design values for the diameter of the filter grain is kept between 1.2 - 1.5 mm and the uniformity coefficient below 1.3.

3.2.5 Chlorination Unit

One chlorination unit with a volume of $1m^3$ is installed above the effluent weir of the clear water reservoir. In Hobigonj, Serajgonj and Gopalgonj it was designed to work by gravity for easy operation. However none of the units is found to work during the site visit.

3.2.6 Clear Water Reservoir(CWR)

The effluent of the rapid filter flows by gravity to the clear water reservoir which has a double function a) storage of clear water for backwashing and b) temporary housing chamber before going to distribution line. The effective volume of clear water tank at Hobigonj, Sirajgonj and Gopalgonj are $151m^3$, $151m^3$ and $135m^3$ respectively.

3.2.7 Sludge Tank

To prevent contamination of paddy field and fields with other crops the backwash water is transported through a sludge tank and a pipe for discharge into a river in the cases of Hobigonj and Sirajgonj. The sludge tank serves as a temporary buffer for the backwash water and is filled under gravity.

Table 3.2 : General Setup of Existing IRPs

Items	Hobigonj	Sirajgonj	Gopalgonj
Capacity	204 m^3/h	204 m^3/h	140 m^3/h
Year of Commission	May 1992	Jan 1991	June1990
No. of Division Chamber	1	1	1
No. of V-notch	2	2	2
Method of Aeration	Cascade (gravity flow)	Cascade (gravity flow)	Cascade (gravity flow)
No. of Cascades	3	3	3
Filter Type	Rapid sand	Rapid sand	Rapid sand
No. of Filter Unit	2	2	2
No. of Chlorination Unit	1*(not working)*	1*(not working)*	1*(not working)*
No. of Highlift Pump	2	2	2
No. of Clear Water Tank	1	1	1
No. of Over Head Tank	1	2 (one left unused)	1
No. of Sludge Tank	1	1	0

Table 3.3: Design Data of Different steps of IRPs.

Items	Sirajgonj	Hobigonj	Gopalgonj
Cascade			
• Number of steps	3	3	3
• Fall height(m)	1.73	1.88	2.13
• Oxygen input (mg/l)	7-8	7-8	7-8
Filter			
• Surface area	13.6m^2	13.6m^2	9.3m^2
• Bed height	1.5m	1.5m	1.5m
• Filtration rate	7.5m/h	7.5m/h	7.5m/h
• Filter medium	sand	sand	sand
• Grain size	1.2-1.5mm	1.2-1.5mm	1.2-1.5mm
• Overflow height	1.8m	1.65m	1.4m
• Backwash rate	40m^3/h/m^2	30m^3/h/m^2	30m^3/h/m^2
Chlorination Unit			
• Dosing tank	1m^3	1m^3	0.8m^3
• Solution strength	0.4-2.4%	0.4-2.4%	0.4-2.4%
• Chlorine dose	2-6 mg/l	2-6 mg/l	2-6 mg/l
Clear Water Tank			
• Effective volume	151m^3	151m^3	135m^3
• Residence time	45 min	45 min	48 min
Sludge Tank Volume	125m^3	146m^3	Nil
High lift/backwash pump			
• High lift capacity	204m^3/h	204m^3/h	190m^3/h
• Backwash capacity	>500m^3/h	>500m^3/h	>345m^3/h

CHAPTER-4

Materials and Methods

4.1 General:

Since the study intends to identify causes of a number of operational problems, review of the hydraulic design of different unit processes of treatment and other operational parameters was done. Documentation of the design information, Physical investigation and close observations of different process, analysis of the each operational step with regard to their efficacy have been the basic approaches of the study.

4.2 Documentation:

In this study design information, drawing and other papers related documents about the background of construction of the IRPs were collected.

An inventory of the existing facilities was also made for a good understanding of plant processes.

Various operational parameters were collected from consultants report. Some operational data like production hours, supply timings, backwashing time, and other activities with respect to pumping, were collected by interviewing the superintendent, treatment plant operators, pump drivers etc.

4.3 Physical Investigation:

Field visits to the plants were also made to investigate them physically. Field visits facilitated the study to make inventory of the plant facilities, better understanding of the process, identify problems and analyze performances of the plants. Water sample collection was made through the field visits.

Some important parameters like CO_2, Fe, DO, turbidity, pH and alkalinity were measured for the raw water and at the various steps in order to determine the effectiveness of the unit process of the plants.

Inflow Quantity:

Overflow weir in form of V-notch is fitted over each filter box. Water from division chamber is distributed to two filter beds through these notches. The quantity of incoming water can be determined by measuring the water depth at the V-notch. The inflow rate can generate some other operational information like, filtration rate, production rate etc.

Filter run:

Filters are running with different durations in the study plants. The runlength were in the range of 7 to 15 hours. During the filter run, hourly headloss was recorded till it reached upto terminal head.

Back washing:

Backwashing is one of the important phenomenon of treatment process as it governs the efficiency of filter units, which in turn affect whole treatment process. Filters were backwashed after each run. The following procedure were followed:

- The CWR was filled with water.
- Level of Filter bed is measured before washing was started.

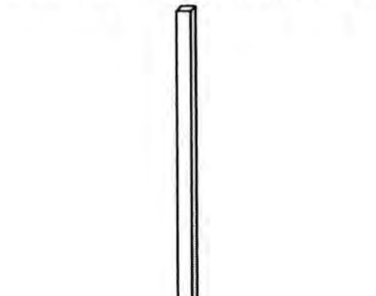


Figure 4.1 Wooden device to determine the expansion of filter bed

- They were washed with clear water by backwash pump.
- They were washed till the CWR became empty. The Volume of water and time required for cleaning is determined. Final level of bed was also

measured to calculate the % of expansion(E) with the help of the equipment as shown in Fig.4.1.

$$E = \frac{L_2 - L_1}{L_1} x100$$

Where L_1 and L_2 is level of filter bed before and after backwash.

- Frequency of backwashing, amount of clean water required & other associated difficulties was recorded.

As it is mentioned earlier that, the plants are running with some operational difficulties, the recurring cost implication of those was also analyzed.

Detailed water quality investigations at different treatment steps including raw water and a laboratory model test were carried out to select appropriate unit processes and to show a comparison study between the field and an ideal condition in the laboratory.

Sample Collection

Water samples were collected from every step of treatment including influent and effluent points of the plants and tested in the BUET Laboratory. Water samples were also collected from the pre-filtered point to examine the performance of filter units.

Water and sand samples were collected in accordance with standard procedure and guidance. Bottles used for collecting water samples were acidified prior to collection. BOD and PET bottles were used to collect water samples.

Total iron, pH, dissolved oxygen(DO), carbon-di-oxide(CO_2), turbidity and alkalinity were measured. Parameters were measured at different points of the plants. Table 4.1 shows parameters measured at different points.

Filter materials:

Filter materials were collected from the different levels of the filter bed for sieve analysis. The results of the sieve analysis is given in appendix-E. The result of the sieve analysis facilitates further analysis of the performance of filtration and backwashing.

Table 4.1: Sampling Points

Collection point	PH	Iron	CO_2	DO	Turbidity	Filter material	Alkalinity
Production well	✔	✔	✔	✔	✔		✔
Cascade-I	✔	✔	✔	✔	✔		✔
Cascade-II	✔	✔	✔	✔	✔		✔
Filter	✔	✔	✔	✔	✔	✔	✔
CWR	✔	✔	✔	✔	✔		✔

Method of Measurement

Parameters were measure according to standard methods by the equipment as mentioned below:

Temperature	Thermometer
DO	Titration
pH	pH meter
Total iron	Titration and iron kit(with standard accuracy test)
Turbidity	Turbidity meter
Alkalinity	Titration
Time	Clock

Besides this, air compressor, 500ml jar, stirring machine, sieve analyzer were also used during the study.

4.4 Laboratory model study with field water samples

Raw water samples from the inlet chamber (division chamber) were collected. The extent of aeration was tested through vigorous shaking and through compressed air for a total time of 5 minutes. Rise of pH and CO_2 was then measured. A sample of 200ml. was then kept unflocculated for a period of 90 minutes. Three samples of 400ml. each were tested for flocculation and sedimentation to find optimum 'G' values with different time. The fourth sample was tested with lime/NaOH. After jar test the samples were allowed to settle and turbudity was measured at 0, 30, 60 and 90 minutes. Optimum 'G' values were then calculated and recorded.

4.5 Analytical Calculations

Water quality parameters like pH, DO, total iron, CO_2, alkalinity and turbidity were measured both in field and in laboratory. pH meter, turbidity meter and in some cases iron meter were used in the field. Other parameters were measured in BUET and DPHE laboratories in Comilla, khulna and Rajshahi.

For other parameter specially hydraulic parameters were calculated with the help of field data and equations:

a) Incoming water discharge through weir (Q):

$Q=1.4h^{5/2}$ m^3/sec (for 90^o V-notch)

Where h = height of water on weir in meter

b) Flocculation basin

Head loss $h = nV_i^2/2g + (n-1)V_2^2/2g$

V_1=velocity of flow between baffles m/s

V_2= velocity of flow at baffles slot m/s(1.5times of V_1)

n= number of bends per comperments

Velocity gradient $G = \sqrt{P/\gamma C}$

Power dissipated $P = Qh\gamma$

c) Sedimentation basin

$$S_o = \frac{q \sin \alpha}{w + t}$$

$$S'_o = S_o \frac{w + t}{H \cos \alpha + w}$$

$$\text{Re ynolds no. } Re = \frac{V_o R}{\upsilon}$$

$$\text{Froud no. } Fr = \frac{V_o^2}{gR} \text{ where } R = \text{hydraulic radius}$$

d) Filtration

$$\mathrm{Re} = \frac{\mathrm{Vfd}}{(1-\mathrm{P})\mu}$$

$$\mathrm{v}^{1.2} = \frac{\mathrm{g}}{130\upsilon^{.8}} \mathrm{x} \frac{\rho\mathrm{f} - \rho\mathrm{w}}{\rho\mathrm{w}} \mathrm{x} \frac{\mathrm{Pe}^3}{(1-\mathrm{Pe})^{.8}} \mathrm{xd}^{1.8}$$

$$\text{where } \mathrm{Pe} = \frac{\mathrm{P}+\mathrm{E}}{1+\mathrm{E}} \text{ and } \upsilon = \frac{497\mathrm{x}10^{-6}}{(\mathrm{t}+42.5)^{1.5}}$$

e) Underdrain

$$\text{headloss} = \frac{\mathrm{V}^2}{2\mathrm{g}} = \frac{1}{6}\frac{\mathrm{v}^2}{\mathrm{n}^2\mathrm{D}_0{}^4}$$

CHAPTER-5

Performance of Existing IRPs : Results and Discussions

5.1 General

The existing IRPs at Sirajgonj and Hobigonj were originally designed and constructed for a treatment capacity of 204 m^3/h, whereas 140 m^3/h for IRP at Gopalgonj. However, the plants are producing 22 to 28% less than the designed capacity for the last several years due to some operational difficulties and treated water is also not meeting the expected (designed) quality of water as was discussed in article-3.2.1. The most important issue is that the plants requires huge quantity of water to clean their filters that it becomes uneconomical to run the water supply system.

This chapter is devoted to describe investigation and analytical results of different unit process, identify problems and point out the causes of the problem in the light of design and operational aspect.

5.2 Selection of Unit Process

Generally water quality parameters govern the choice of unit process of treatment. The table 5.1 shows that there is a distinct variation of raw water quality parameters between the IRPs under study. Sirajgonj and Gopalgonj IRPs have very high iron content of above 8 ppm and Hobigonj has moderately high content of 4.5ppm. Alkalinity, which plays an important role in oxidation of iron also varies widely. Hobigonj has low alkalinity of 152ppm and Gopalgonj has 497ppm. CO_2 content also varies from 50 to 130ppm.

In spite of large variation of water quality parameters a prototype unit process and design was selected for the towns under study.

The water quality parameters after aeration, as described in the table 5.1 shows that iron precipitation rate is not sufficient at Hobigonj IRP due to low alkalinity. Release of CO_2 and subsequent rise of pH was occurred almost at the same rate. This is about 50% decrease and increase respectively. As a result CO_2 at Sirajgonj IRP decrease from 50 to 20 and at the same rate CO_2 in Gopalgonj IRP reduced from 130 to 70ppm. This means that Gopalgonj IRP was required extra treatment to reduce CO_2 content further. Similarly, sufficient increase of alkalinity was not observed in Hobigonj IRP. So this water requires extra treatment to increase alkalinity.

Moreover, Table 5.1 also reveals that the raw water quality parameters do not qualify for direct filtration. The recommended parameters as made by AWWA is presented in Table 5.9. It is evident from the discussion above that selection of unit process was not correct and it would have been made on the basis of raw water quality of individual towns.

Table: 5.1 Water Quality Parameter at Different Steps of Treatment process.

Sl. no.	Parameter	Sirajgonj	Hobigonj	Gopalgonj
	Raw water			
	Total Iron (ppm)	8.7	4.5	8.0
	Alkalinity (ppm)	250	152.5	497
	PH	6.9	7.1	7.0
	CO_2 (ppm)	50	90	130
	Temperature (^{0}C)	24	22	24
	Turbidity (NTU)	1.15	1.25	1.25
	Aerated water			
	Alkalinity (ppm)	225	155	501
	PH	7.3	7.75	7.5
	Co_2 (ppm)	20	55	70
	Temperature (^{0}C)	26	25	25
	Tubidity (NTU)	32	10	28
	Filtered water			
	Total Iron (ppm)	0.7	0.3-0.4	0.6-0.8
	Alkalinity (ppm)	225	150	487
	PH	7.4	7.7	7.7
	Temperature (^{0}C)	28	27	29
	Turbidity (NTU)	1.0	4	1-2

5.3 Aeration:

Performances of the aerators of the study plants have been studied in terms of adsorption of oxygen, desorption/removal of carbon dioxide and change in pH values. The detail of water quality test at different steps of the cascade is presented in table 5.7 later in this chapter.

5.3.1 Extent of Oxygen Uptake:

The oxygen concentrations of raw water were measured at division chamber, at different steps of cascade and filter. Theoretical oxygen concentrations were also calculated according to a method given by *Popel & Post*. They derived a formula to find out oxygen transfer by cascades as follows:

$C_e = C_o + K(C_s - C_o)$.

Where '*K*' is efficiency coefficient which maintains linear relationship with weir height upto 0.64m and,

C_O, C_e and C_s are concentrations at initial, effluent and saturation level respectively.

Above the height the influence of *h* on *K* is less pronounced. C_s-C_O act as the driving force of oxygenation. For higher C_s-C_O value faster oxygenation is occurred.

Ideally dissolve oxygen concentration in ground water is zero, however, during transportation through tubewell and during fall at the division chamber water gained some oxygen. In the calculations this concentration is taken as raw water concentration. A comparison between theoretical uptake and observed concentration at different steps of aeration is given in table 5.1 for the three study towns.

Table 5.2: Oxygen uptake at different steps and a comparison with theoretical uptake.

Plant	Oxygen concentrati on	Division chamber	Cascade steps ; figures are in mg/l			Pre-Filter (1.3m)
		Raw	1st step 0.5m	2nd (0.5m)	3rd 0.33m)...	
Sirajgonj	Observed	3.95(47%)	5.10(61%)	5.91(70%)	6.55(78%)	7.20(85%)
	Theoretical	3.95(47%)	5.80(69%)	6.88(82%)	7.27(86%)	8.06(96%)
Hobigonj	Observed	2.20(26%)	4.51(54%)	5.98(71%)	6.69(80%)	6.70(80%)
	Theoretical	2.20(26%)	4.78(57%)	6.28(75%)	6.83(81%)	7.8(93%)
Gopalgonj	Observed	3.10(37%)	4.96(59%)	6.38(76%)	6.87(82%)	7.00(83%)
	Theoretical	3.10(37%)	5.30(63%)	6.59(78%)	7.06(84%)	7.99(95%)

It is evident from the Table 5.2 that initial oxygenation rate was faster in Hobigonj, it is because of comparatively high driving force (c_s-c_O). However, the oxygen uptake at the end of aeration i.e. on the filter bed of Sirajgonj, Hobigonj and Gopalgonj were measured at 7.20, 6.70, & 7.00 ppm respectively against a saturation value of 8.4 mg/l at 25°C. The corresponding uptake in percentage of saturation were 85, 79 and 83%. Attempt has also been made to optimize the oxygen uptake through laboratory model test. Model test shows that concentration can be increased upto 92, 94 and 96% respectively through extensive shake and passing compressed air through raw water.

According to stoichiometric relation 0.14 mg/l of oxygen is required to oxidize 1 mg of Fe^{++}. Using stoichiometric relation the oxygen requirement of oxidizing the Fe^{2+} are 1.2, 0.63 and 0.59 ppm respectively. That is the oxygen gained at the division chamber is sufficient for oxygenation of Fe^{2+} and further aeration by cascade is not necessary. However, according to Degremont, the rate of oxidation is dependent on dissolved oxygen content as described in literature review. Normally 70% saturation is considered to be sufficient for successful oxidation of iron for removal.

So, it can be stated from the above discussion that the oxygen uptake in the plant are quite sufficient.
It is also evident from the Table 5.2 that desired oxygenation has been occurred at the 3rd step of the aerator which provides a fall of 1.33m against a total fall of 2.13m which means that the total height could have been optimized which would save not only the capital cost but also the pumping cost.

The study attempted with different height 1.50, 1.25 and 1.00m to find out the optimum height and number of steps required. The results are presented in Table 5.3. Calculation shows that two step cascades are giving maximum oxygenation at each height i.e. 1.5, 1.25 and 1.0m. 1.5m fall height gives maximum oxygenation at two step cascade, however, oxygenation for height 1.25 is close to that of height 1.5m. 1.25m cascade may be selected on economic consideration and to be on safe side of oxygenation. The optimization of cascade could reduce not only the construction cost but also the electricity cost for pumping as fall height is decreased.

Table 5.3: Optimization of cascade fall by theoretical uptake calculations.

Town	Fall height	Cascade Types (figures are in ppm)		
		1step	2 step	3step
Sirajgonj C_o=3.95ppm	H=1.5m	7.64(90%)	7.65(91%)	7.48(89%)
	H=1.25m	7.37(87%)	7.39(88%)	7.17(85%)
	H=1.00m	6.97(83%)	6.9(82%)	6.59(78%)
Hobigonj Co=2.2ppm	H=1.50m	7.30(86%)	7.36(87%)	7.16(85%)
	H=1.25m	6.97(82%)	7.39(83%)	7.17(79%)
	H=1.00m	6.40(76%)	6.31(75%)	5.89(70%)
Gopalgonj Co=3.1ppm	H=1.50m	7.50(89%)	7.53(90%)	7.31(87%)
	H=1.25m	7.18(85%)	7.19(85%)	6.90(82%)
	H=1.00m	6.7(79%)	6.9(82%)	6.25(74%)

5.3.2 Release of CO_2

Desorption of CO_2 occurred simultaneously to adsorption of O_2 as the water passes through the aerator. CO_2 is removed from 50, 90, 130ppm to 20, 55 and 75ppm in Sirajgonj, Hobigonj and Gopalgonj respectively. The percent removal of CO_2 is 60%, 39% & 42%. The removal of CO_2 is important in the removal process of iron as it is accomplished with increase of pH value.

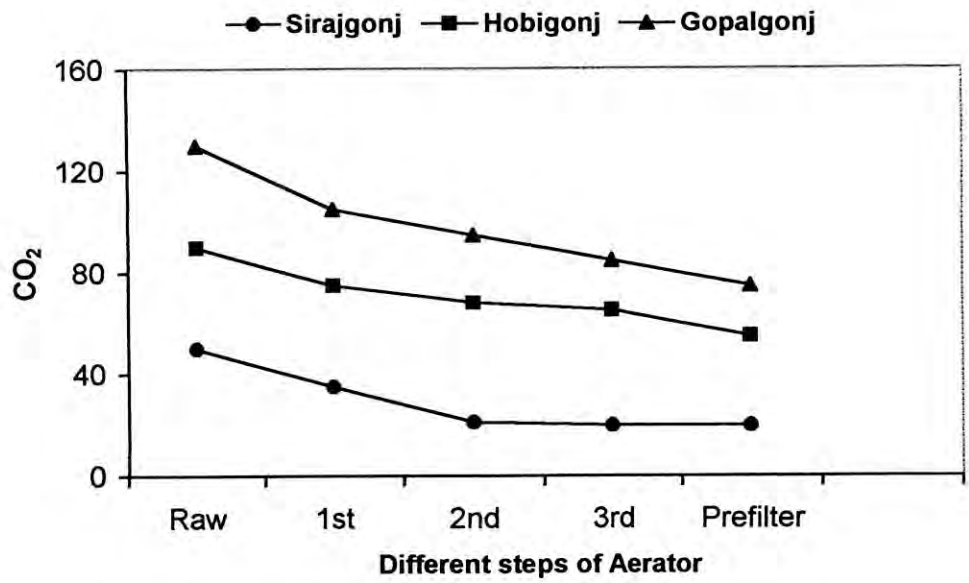


Figure 5.1: Release of Carbon dioxide in Aerators

5.3.3 Increase of pH value:

It is seen from the figure 5.2 that pH is increased upto 7.4, 7.7 and 7.5 in Sirajgonj, Hobigonj and Gopalgonj respectively through existing aerator, however, Laboratory model test shows that pH can be increased upto 7.9-8.1, 8.2 and 7.9 respectively by extensive shaking and passing compressed air through the raw waters.

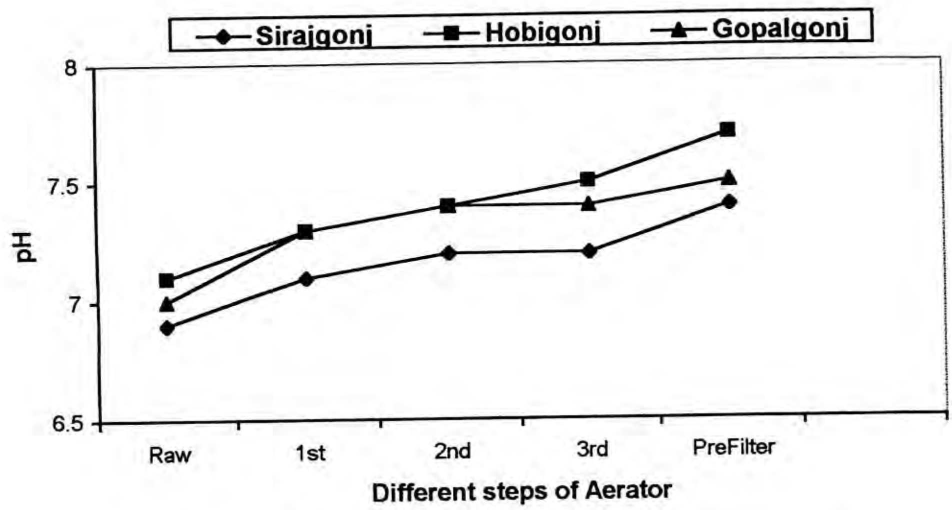


Figure: 5.2 Increase of pH value in Aerator steps

While reviewing the kinetics of iron oxidation it was found that in solution with pH>5.5 the rate of oxidation is of the first order with respect to both Fe^{+2} and O_2 concentration and second order with respect to OH ion i.e. the oxidation rates increase 100 fold for an increase of one pH unit. Another study in IHE, Delft shows that in aeration flirtation process, optimal pH for iron removal is <7 and headloss development is a function of pH (*Adekoya, 1994*).

It has been observed in the previous article that oxygen uptake of the plants under study is close to saturation level which requires extensive aeration leading to an markable increase in pH by desorption of CO_2 from water. The marked rise in pH causes rapid oxidation in solution forming floc precipitates over the bed surface as has been observed during field study. This in turn creates surface mat and cause short filter run.

On the other hand, had there been no extensive aeration significant rise in pH would not occur. In such case deep bed filtration would occur at pH less<7. For the later case filter run length would increase due to less head development. However there are risks of deterioration the effluent water quality.

Another point which can be mentioned here that, the filter run in Hobigonj is higher than that of other two towns despite the fact that increase of pH in this town is higher. It is because that the Fe^{+2} and O_2 concentrations are comparatively less and so, iron is not oxidized at sufficient rate and precipitated over the bed.

Overview:

- Water quality investigation and model tests were not carried out before selection of unit processes of treatment for the existing IRPs. Prototype design was made for all towns.
- Oxygenation is sufficient in cascades.
- As 70% oxygenation is sufficient for oxidation of iron, optimization of cascade fall could have been done.
- Increase of pH was so sufficient to cause rapid oxygenation leading to surface mat over filter bed surface.
- Flock precipitates on filter bed led to short filter run.
- pH rise below 7 cause deep bed filtration and filter run length would increase, however, risk left for deterioration of effluent water quality.
- Optimization number of cascade and fall height is possible. Calculations show that 2 step will fall of 1.25m is optimum.

5.4 Filtration Process

The general description of the filters is given in chapter-3. All the filters are rapid sand filters having constant rate of filtration. In this section the effects of size of filter materials, position of wash water trough, cleaning of filters, qualification of raw water for direct filtration and under drainage system on performance of treatment plant are reviewed. In addition efficacy of present backwashing and its cost implication is also discussed.

5.4.1 Filter materials

Samples of filter materials have been collected from study plants. Careful sieve analysis were carried out in DPHE R&D Division's laboratory of the collected samples and their results are given in table 5.3. The table also presented a Comparison of different parameter of filter material with that of design values. The filter materials used in the plants are natural sand with 1.5m in depth in single layer. It was found that the grain size distribution of applied filtering materials are not in accordance with the specification of the commonly practiced values.

Table 5.4 Filter Material Parameters used in the Existing Plants.

Parameter	Sirajgonj	Hobigonj	Gopalgonj	Design
d_{10}(mm)	0.6	1.00	0.9	1.2
d_{60}(mm)	1.35	1.45	1.40	1.5
Size range, (mm)	0.6-1.65	1-1.70	0.9-1.75	1.2 – 1.5
U_c	2.25	1.45	1.55	<1.3

The grain size parameters in the IRPs under study differ from that in the design values. The obtained uniformity coefficient(U_c) for all the towns have higher values than that recommended (i.e. 1.3 or less). The filtering material in Sirajgonj plant can easily be termed as non-uniform grains as its U_c is far above the recommended value. The effective size is also below the minimum recommended size 0.8mm.

As explained in Article 2.7.4, filters are cleaned by shearing action of the rising wash water stream flowing at high rates past the stationary grains. This shearing force(τ) is directly proportional to diameter (d) of the grains:

$$\tau = d/6(\rho_f - \rho_w)$$

So when the grain size of the media is smaller the resultant shearing force τ becomes so small to cause effective cleaning. Another experiment stated that sand grains with a diameter less than 0.8mm are difficult to keep clean by backwash with water alone.(*Huisman, 1986*)

In Sirajgonj cleaning efficiency is very poor. Mudball and thick claylike iron sludge blanket is observed at the media surface. Huge sticky iron flocs were seen even after backwashing. It happens because of non-uniform filtering materials where backwashing results in stratification, with fine grains in the upper and the coarse grain in the lower part of the filter bed. Backwashing at low rate expands the upper part while in the lower part the grains remain stationary. Thus hampering removal of impurities accumulated during previous filtration. Filter cracks are also developed due to fine grains. It causes short circuiting and deteriorates effluent water quality.

Table 5.5 supported the fact as described above and shows that Sirajgonj plant requires highest amount of water for backwashing.

5.4.2 Correlation of Filtration rate, Terminal head, Filter-material and Influent iron Concentration with Run length.

Runlength depends directly on terminal head, depth of filter bed, grain size and porosity and inversely on filtration rate and influent concentration. Higher runlength is observed in case of Hobigonj with lower filtration rate and higher terminal head. At Gopalgonj IRP, though filtration rate is lower but higher runlength is not observed. It is due to the fact that terminal head is less and iron content in raw water is higher.

Table 5.5: Correlation of Filtration rate, Terminal head, Grain size and Influent Iron with Length of Run.

Parameter	Sirajgonj	Hobigonj	Gopalgonj
Length of Run *(h)*	7-9	14	6-7
Filtration rate *(m/h)*	6.3	5.92	5.4
Terminal head *(m)**	1.80	1.65	1.40
Grainsize *(mm)*	0.6	1	0.9
Influent iron *(ppm)*	8.7	4.5	8.00

5.4.3 Variation of Filter Head loss

The headloss vs time curve as presented below reveals that the headloss increased sharply during first two hours. Then the curve tend to be flat and become stable for few hours and then rise sharply again.

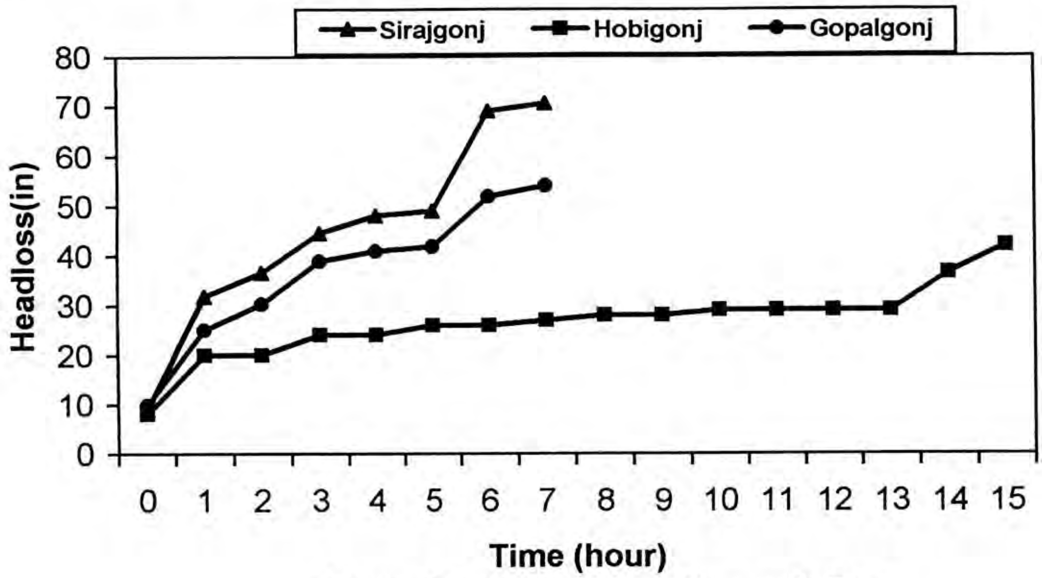


Figure 5.3 Change of Headloss with Time

The rapid headloss during first two hours was occurred due to precipitation of oxidized iron which formed surface cake on the filter bed and it consumed nearly 50% of usable head within first two hours.

The remaining 50% of the head is left to be used for next several hours. During this time linear or tend to be linear headloss development was observed. It is because, risen water column compressed the sludge mat and leading the filtration into deep penetration. During this time the curves exhibit a tendency to become flat.

After some time when pore space available are begun to fill up by sticky iron flocks, sharp increase of headloss occurs which eventually leads to overflow.

5.4.4 Backwashing Process

The filters of Sirajgonj and Gopalgonj plants are washed twice a day (in the morning and in the evening) and the filter of Hobigonj is washed once a day, in the evening. The backwashing is being performed with a backwash rate of 32, 37 and 34 m/h in Sirajgonj, Hobigonj and Gopalgonj respectively. The amount of backwash water in those three towns are 623m^3, 341.34m^3 and 442m^3 per day which accounts for 24, 14 and 31% of the treated water respectively. Table 5.6 revealed that amount of backwash water required for

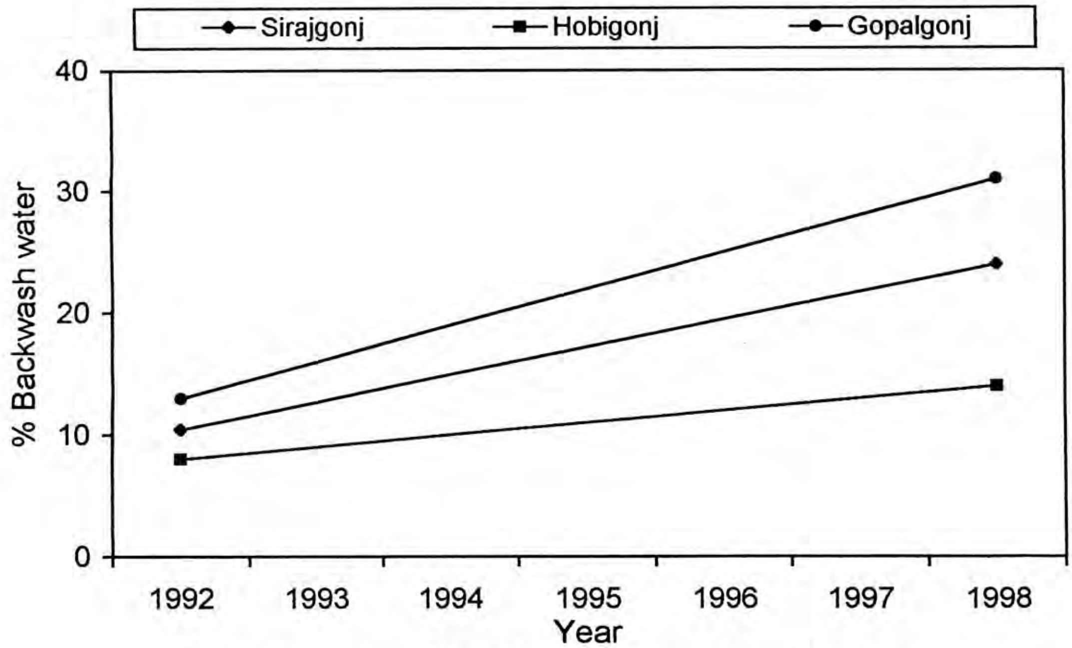


Fig: 5.4 Amount of Washwater Increasing with Time

the study plants with direct filtration process is abnormally high, In spite of huge amount of wash water, the study found that the level of washing is also not satisfactory. It was found that the turbidity of overflowing water at the last moments of backwashing reached to 30 to 37ppm. It can be be mentioned here that the maximum recommended value is 10 NTU.

However, at Faridpur IRP with conventional treatment process, only 4% water is required which is within the recommended value.

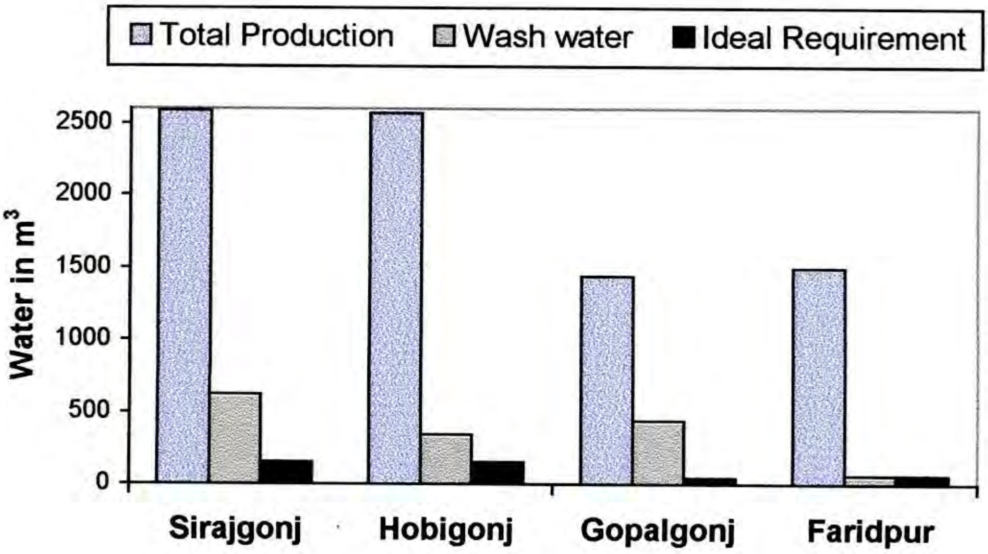


Figure 5.5 Amount of Washwater and Water production

The details of backwashing particulars and calculations are presented in Appendix-A. The study also found that the amount of washwater as percent of total production has been increasing sharply since the commencement of the plants.

The common practice in rapid sand filtration recommends that the amount of backwash water should be limited to between 1 to 5% of the treated water. For direct filtration the amount could be a little bit higher upto 6% as compared with 3-4% for conventional plant. But 14 to 31% water requirement indicates design and operational deficiencies of the plants.

Table 5.6 Amount of Backwashwater required for Different Plants

Location	Filter Area m^2	Washing time (min) (per filter)	Washing rate (m^3/h)	Total Amount (m^3/day)	Backwash water (%)	Filter run (hour)
Sirajgonj	13.6	40	467	623	24	7-8
Hobigonj	13.6	20	512	342	14	12
Gopalgonj	9.3	40	331	442	31	7-8
Faridpur	20	5	60	60	4	31

The study thus conducted thorough investigation in order to find the causes for such poor performances and identified several design and operational deficiencies which are discussed in the following sections:

5.4.5 Collection of Wash-water

Both Washing of the filter bed and collection of wash water were not uniform as has been observed during study. Physical observations also shows that washing is poor at the distant points from the trough. The measured turbidity at the end of washing is about 30 to 40NTU. The study considered that the wrong placement of trough is one of the major cause for which uneven and poor washing of the beds was occurred and thus resulted in requirements of huge quantity of water for cleaning.

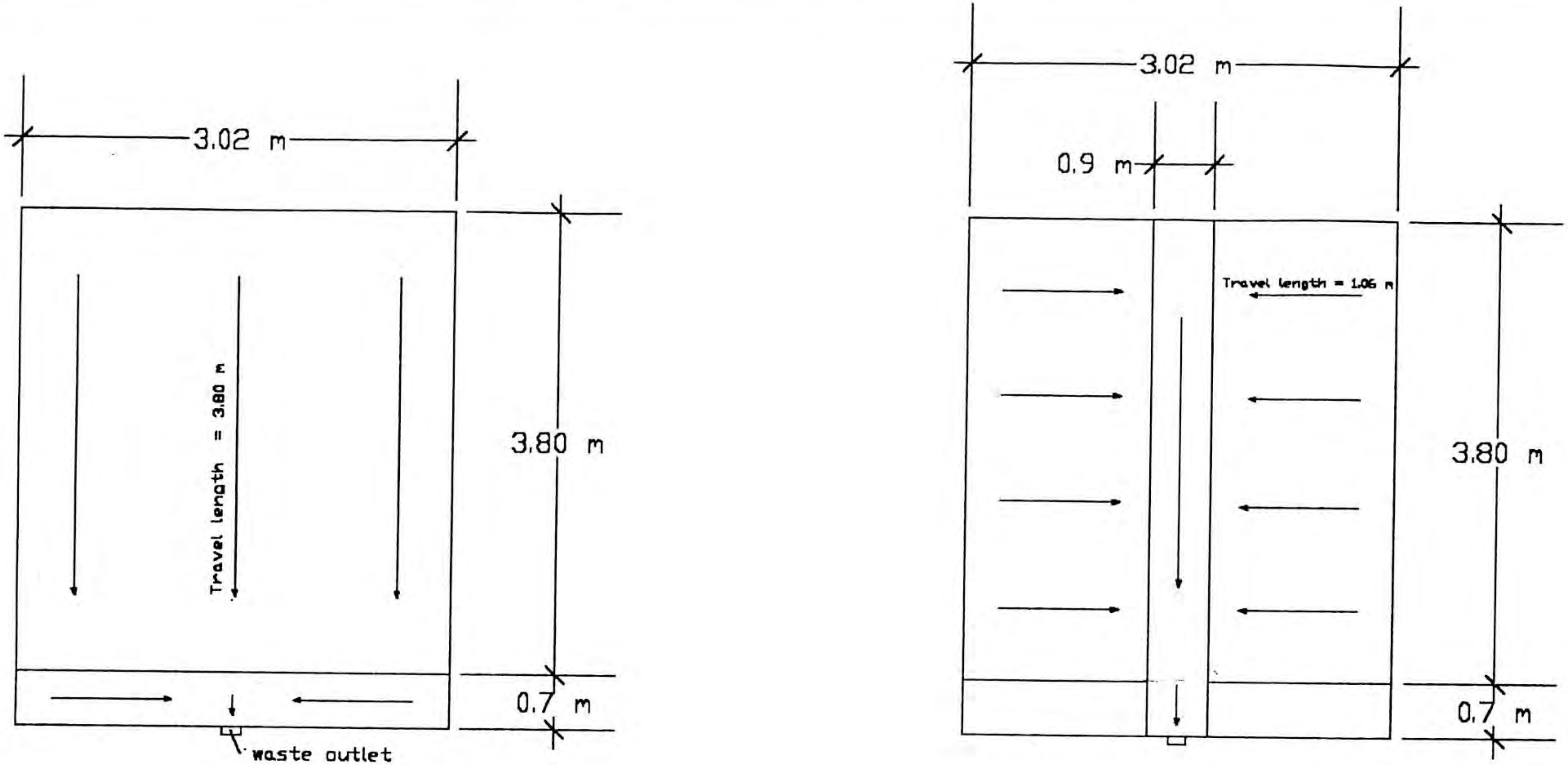
In all the plants the trough run the width of the filter and was attached to the shorter wall at an elevation of 1.15m from bed top. Since the length of the filter was 3.8m the lateral travel of the water overflowing into the trough is 3.8m. Both the lateral distance and vertical height from the bed top appears to be higher. So the impurities while travelling longer distance of 3.28m resettle down the bed before it reaches to the trough. According to American practice and different literature the lateral and vertical distance should be limited to 1.06m and 0.9m respectively.(*AWWA, 1995*)

Table 5.7: Comparison between Designed and Required size of Wash Trough.

	Designed	Recommended Limit	Required
Height of Upper edge (from unexpanded bed)	1.15m (0%expansion)	0.9m	0.45m(for 0% expansion) 0.70(for 20% expansion)
Lateral travel	3.28m	1.06	1.06m

Generally, the upper overflow edge of the washwater trough are placed sufficiently near to the surface of sand so that most of the washed out impurities are removed easily in short time .

On the other hand, this upper edge should be set at a minimum distance of 0.25m above the top of expanded sand to prevent loss of sand during washing. A minimum distance of 0.05m is also kept for the bottom of trough above the above the expanded sand bed.



EXISTING WASHWATER TROUGH
(Sirajgonj for example)

MODIFIED WASHWATER TROUGH
(Detailed in appendix-4)

Figure 5.6 : Existing and Modified Washwater Trough

Since the depth of the sand bed is 1.5m in the IRPs under study, a provision of 20% expansion of the filter bed requires a minimum vertical distance between the upper edge of the trough and the unexpanded sand bed should be set at about 0.60m [0.3m(expansion)+ 0.25m(upper edge)≈0.6m]. The existing distance is much higher than recommended value of 0.9m, caused partial removal of impurities from the filter.

The troughs could have been placed in the middle of the filter bed parallel to its width as shown in figure 5.6. In that case the lateral travel would be 1.16m. This arrangement would result a effective cleaning, nearly uniform wash and required less amount of backwash water.

5.4.6 Insufficient Bed Expansion:

In design review of the plants it is found that the filter bed has designed to expand at 0%. However, the study during operational investigation found that the bed expanded only 0.92-1.62%.

Table 5.8 Expansion of Filter bed

Location	Backwash Rate(m/h)		Expansion of filter bed		
	Actual	Required	Design (%)	Existing (%)	Desired (%)
Sirajgonj	32	54	0	0-1.01	20-25
Hobigonj	37	54	0	0-0.92	20-25
Gopalgonj	34	54	0	0-1.62	20-25

As a matter of fact, water-alone Backwashing is inherently a weak cleaning process. Experiments shows that unless hydrodynamic shear by particle collision is created through expansion of filter bed by 20-30% desired level of cleaning could not be achieved (Amirtherajah, 1978). This study has calculated the required washing rate for 20% bed expansion and has described in table 5.8. The detailed calculations are given in Appendix-C. The table 5.8 revealed that the filters in the study plants are being backwashed at lower rates than required. Since the filters under study are being washed at insignificant expansion, it results insufficient particle collision. Those collisions refer to hydrodynamic shear on the grain surface, which is essential for effective cleaning. In the literature review it has been mentioned that, using water alone for backwashing is inherently weaker process and require about 25% expansion of filter bed.

During study attempts were made to clean the filters effectively. Since it was found that the bed was not expanded or loosened vigorous shaking by wooden stirrer was made. Some quantity of raw water was allowed to come through V-notch to make cross cleaning by dilution. *It was found that better wash was achieved (turbidity 15-20) 5 min. before normal wash. It implies that about 25% water can be saved from existing backwash practice.*

It may be concluded here that this small bed expansion for lower backwash rate is the principal cause for the incomplete bed washing.

Method of backwashing: Backwashing is being performed by high lift pumps at 10m head which was providing a wash rate of 32 to 37m/h. However, at present the rate is decreasing as given in table 5.8. The pump efficiency is decreasing with time. So constant wash rate cannot be maintained by pumps. Backwashing by OHT could minimize this problem by providing constant head.

5.4.7 Recommended Raw water Quality for direct filtration:

The American Water Works Association(AWWA) has nominated the source water having following criteria as a 'Perfect Candidate' for direct filtration*(Water Quality & Treatment, 1990)*:

Table 5.9 Comparison of Recommended & Actual Quality of Water for Direct Filtration

Parameter	Recommended	Actual
Iron	< 0.3 ppm	4.5-8.7ppm
Manganese	< 0.05 ppm	nm
Color	< 40 CU	nm
Turbidity	< 5 NTU	32,28,10
Algae	< 2000 asu/ml	no trace

Cleasby et al., however, suggested turbidity limits of 12 NTU when alum is used alone. The above recommendation are made based on the pilot or full scale observations. Table 5.9 shows the water quality suitable for direct filtration in the study towns. The concentrations revealed that the raw water does not qualify for direct filtration.

So, it is evident that un-permitted, unsuitable water is being used in direct filtration in the study plants which results short filter run and huge backwash water requirement.

5.4.8 Under Drainage

The study plants use PVC made media retaining nozzles for filter underdrain. The slot height and width is 46mm and 0.5mm respectively with 36 slots per nozzle and 47 such nozzles per square meter of the filter. This gives a ratio of total opening area of slot to total area of filter $0.027m^2$. This ratio is approximately ten times higher than that of commonly used in lateral-manifold system (0.0015-$0.005m^2$). Such opening provide a $1/10^{th}$ lower velocity of water from the nozzles during the period of washing.

The study considered that the selection of this nozzle type under-drainage system was wrong and this system is responsible for ineffective and poor cleaning of the filters. The reasons are:

a) Since the nozzles are distributing wash water to the bed with low velocity it fails to achieve the main objective of the underdrainge system. In the literature review it was mentioned that the responsibility of under drainage system is to breakup, loosen and remove incrustant accumulated on filter grains and to breakup mud balls formed.

   Mud ball were found not to be removed or broke up. Incrustants are not completely loosen and removed.

b) Pre-treatment process is absent in the study plants and the raw water contains high iron concentration. As a result nozzles working with low velocity subject to frequent clogging of its openings.

c) The nozzle is an imported item. It is not available in local market so valuable foreign currency will have to be spent in case of replacement.

The study also considered that most commonly used underdrain system, the lateral-manifold, could have been selected in the plants. A lateral-manifold system has a jet action resulting from high velocities from the perforations during backwashing. There is a lower possibility to clogging the perforation by iron deposits due to this jetting action. This type has another merit. It can be constructed locally with locally available materials meaning save of money and ease of O&M.

In Europe and USA filter underdrains are reported to have been failed due to clogging of the small sized slots and this type of underdrain is strongly discouraged to use in direct filtration (*Schipper, 1996*).

5.4.9 Filter Efficiency

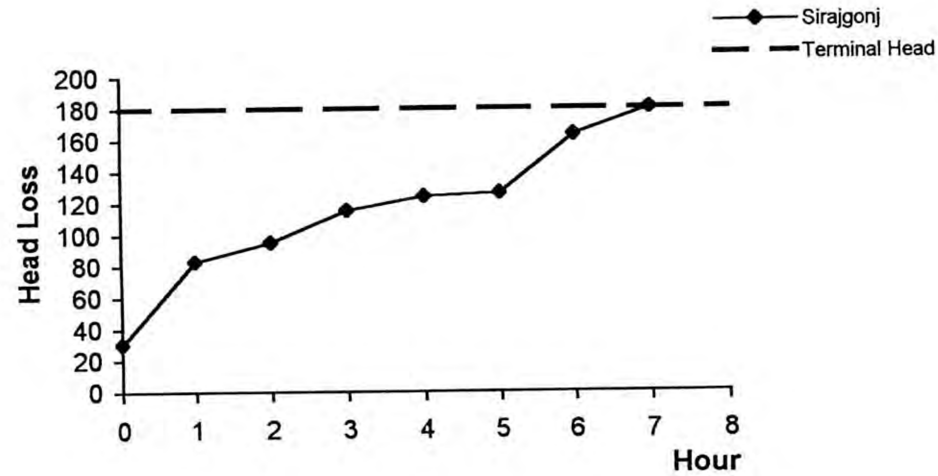
Hourly effluent iron concentration was measured at the plant site with iron meter. Effluent iron concentration vs. time curve was drawn which is shown Figure 5.7.

The time when the filter head loss reach to overflow level i.e. terminal head is indicated by T_h and the time when breakthrough of effluent water occurs is indicated by T_q in the Figure 5.7. According to design, overflow would have been occurred before the breakthrough of iron occurred i.e. $T_q > T_h$. This condition is also desired and practiced commonly in order to ensure effluent quality.

But practically opposite picture is observed $T_q < T_h$ in the IRPs. However, Optimization of filter design demands that at least T_q should at least equal to T_h (T_q=T_h). In the optimization of filtration process, parameters like, rate of filtration, grain size, filter depth and influent Conc. etc. are important. Selection of proper combination of grain size and filtration rate with several trial may results such optimization.

The figure 5.7-5.9 reveals that break through of iron occurs much before the overflow occurs. Actually the effluent of iron concentration in the IRPs under study do not meet neither the WHO standard nor the design standard for effluent iron concentration.

Sirajgonj



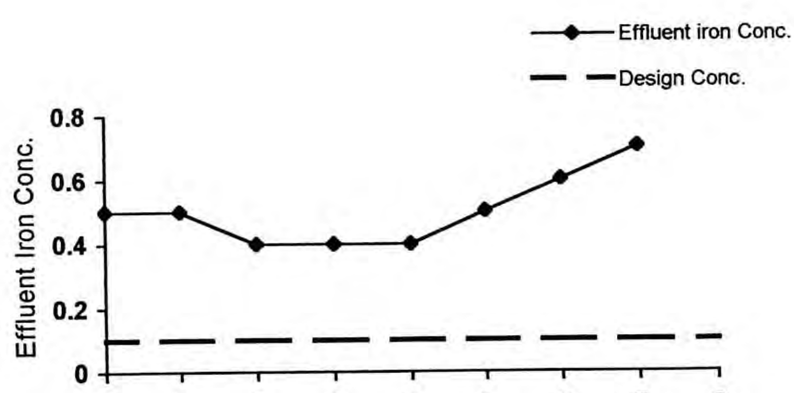
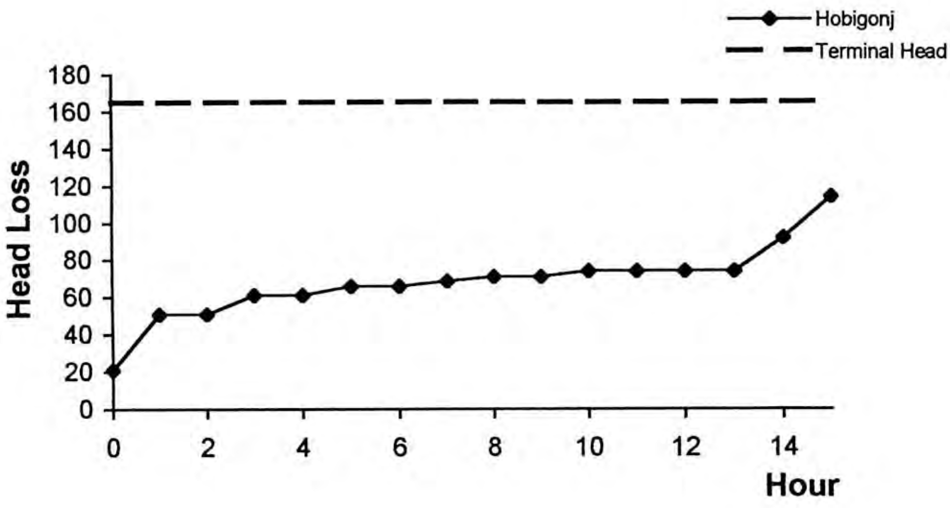


Figure 5.7: Length of Run with respect to Head loss and Effluent Quality(T_h, T_q) in IRP at Sirajgonj

Hobigonj



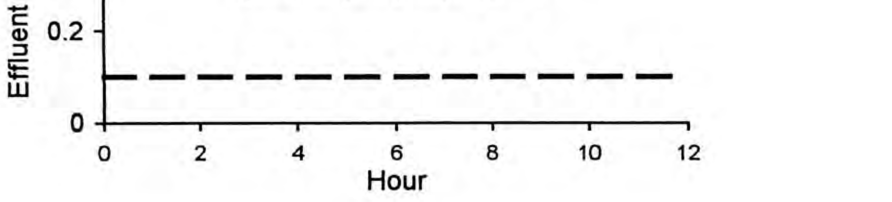
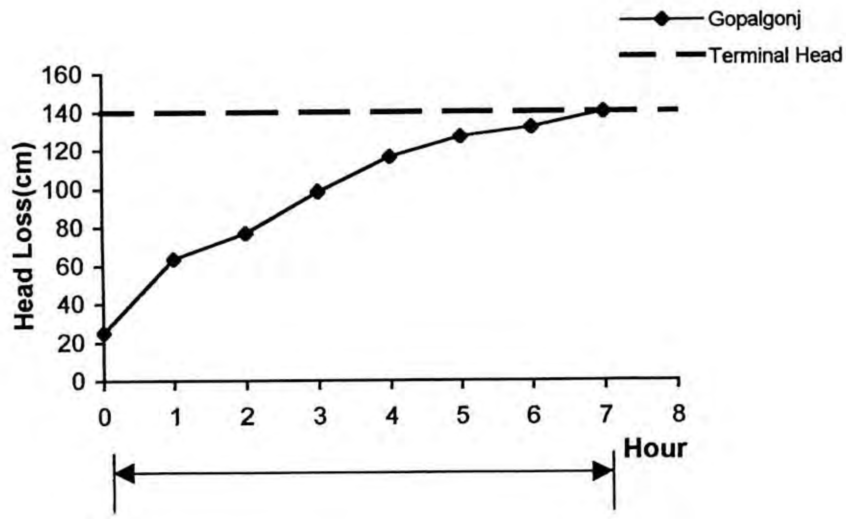


Figure 5.8: Length of Run with respect to Head loss and Effluent Quality(T_h, T_q) in IRP at Hobigonj

Gopalgonj



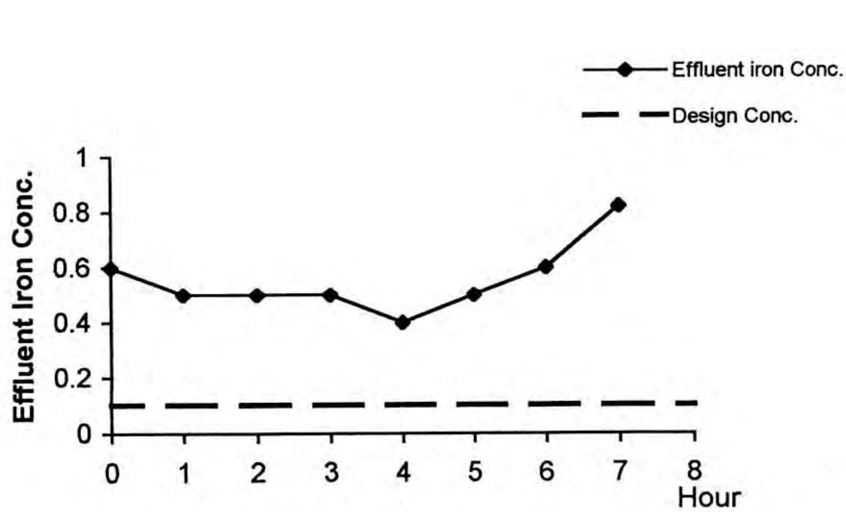


Figure 5.9: Length of Run with respect to Head loss and Effluent Quality(T_h, T_q) in IRP at Gopalgonj

Table 5.10: Variation of Ce/C_o values with time

Plant (Ce/Co)	Hour							Average %
	1	2	3	4	5	6	7	
Sirajgonj Ce ppm C_e/C_o	0.5 5.75%	0.4 4.6%	0.4 4.6%	0.4 4.6%	0.5 5.75%	0.6 6.9%	0.7 8%	5.74
Gopalgonj Ce ppm C/C_o	0.6 7.5%	0.5 6.25%	0.5 6.25%	0.5 6.25%	0.5 6.25%	0.6 7.5%	0.82 10.25%	7.17

Plant	Hour												Avg.
	1	2	3	4	5	6	7	8	9	10	11	12	
Hobigonj C_e/C_o	11.11 %	10 %	8.88 %	6.67 %	6.67 %	6.67 %	6.67 %	6.67 %	6.67 %	7.77 %	8.88 %	10 %	8.05

Sirajgonj: The table 5.1 for water quality parameter shows that the iron concentration in raw water is 8.7ppm in Sirajgonj. The ratio of hourly effluent concentration to the raw water concentration(Ce/Co) is calculated and given in table 5.10. From that table the average value of C/C_o is found to be 5.74%. If we set a design value of effluent iron concentration 0.1ppm then the filter can be used to treat a maximum of 2ppm of iron. **This implies that the Sirajgonj plant is taking more than four(4) times of iron load than it can handle.** This condition is presented in Table 5.11.

Table 5.11: Extent of iron load on the Filters in the IRPs

Towns	Influent iron (ppm)	Filter capacity to treat (ppm)	Designed effluent, (ppm)	Filters Overloaded by
Sirajgonj	8.7	2.0	0.1	4 fold
Hobigonj	4.5	1.4	0.1	3.5 fold
Gopalgonj	8.0	1.5	0.1	5 fold

Gopalgonj: The average value of C_e/C_o is calculated and found to be 7.17% from the table 5.10. If we set a design value of effluent iron concentration 0.1ppm then the filter can be used to treat a maximum of 1.5ppm of iron. **This**

implies that the Gopalgonj plant is taking more than five (5) times of iron load than it can handle.

Hobigonj: The average value of C/C_o is found 8.05% from the table 5.109. The design value of effluent iron concentration is 0.1ppm then the filter can be used to treat a maximum of 1.4ppm of iron. **This implies that the Hobigonj IRP is taking 3.5 times of iron load than it can handle.**

5.4.10 Filter's Economic Life:

The overall effect in all the treatment plants is that the filter runlength is decreasing with time from 10, 16 and 17hr in 1992 to 8, 12 and 7 hr. in 1998 in the study plants as shown in fig: 5.8. One of the reasons behind it is that the iron concentration in the inflow has been increased much more than that at the time of commencement. But more importantly, incomplete backwash, surface mat formation is the cycle cause that bringing down the run length of the filters in the course of time. Moreover, the study found some design loophole as already been discussed, which cause such deterioration of performance.

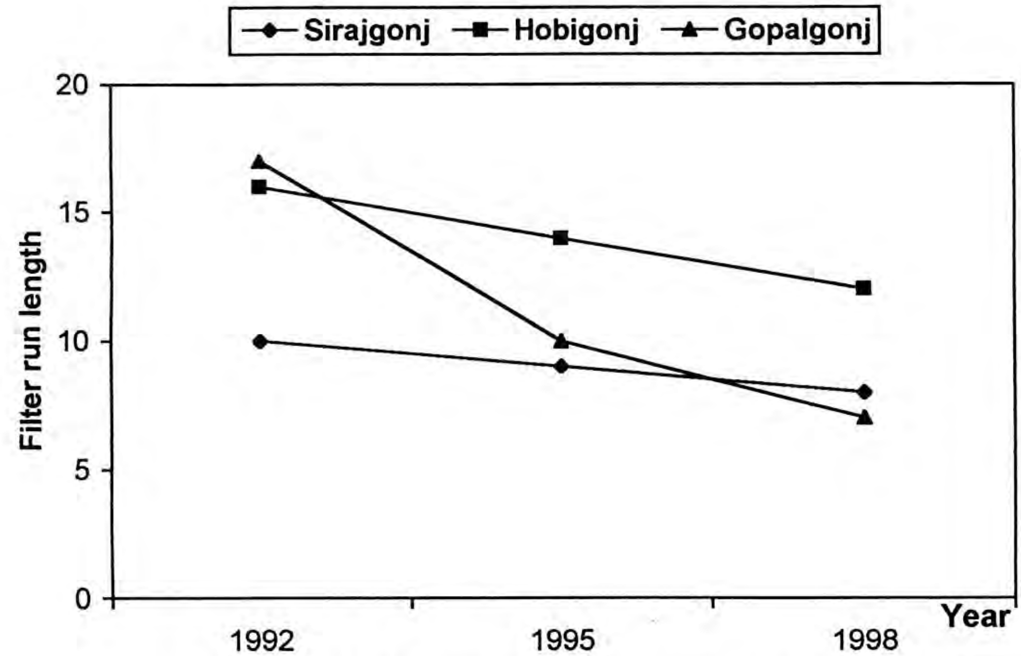


Figure 5.10: Change of Filter runlength with Time

5.5 Operation & Maintenance:

Operation and maintenance activities are difficult in the plants. This problem is found to be inherent in all the plants. The plants are duplex in type. Pump unit is located in the ground floor. Filter unit and control panel is in the first floor however they are installed in different elevation. Chlorinating and clear water reservoir though located the same floor but one has to step up or down to reaches those units through stair.

The major routine activities in the plant are: Pump switch on/off, V-notch open/close, observation of headloss in the filter units, control of chlorination unit, watch on control panel etc. All these activities are to perform manually. So the pump driver has to move himself to various units located at different location and levels.

During field investigation it was found that log books were not maintained with regard to detail of pump operation and backwashing. The driver on a shift did not know how many hours the pumps run or if there was any difficulties occurred during previous shift. For want of the log book it was also not possible to know which filter was washed and how much time was taken by that filter etc. Besides these, inflow rate, distribution period, backwash time and frequency were not recorded for evaluation or monitoring.

It was also observed that there was no arrangement for air circulation through fan over filter beds. Air circulation is necessary specially in Sirajgonj and Gopalgonj as the raw water there contains foul gases.

Besides the above rapid accumulation of iron sludge on filter bed, measuring scale and filter walls demand frequent cleaning. During the study it is found that in Hobigonj cascades and filter top were cleaned once in a week. Accumulation rate is faster in Sirajgonj and Gopalgonj, nevertheless cleaning is done once in a month. The pump driver stated that sufficient budget for cleaning and other maintenance work was not available.

The most unbearable problem is the high operation & Maintenance cost of the plant. From the table 5.5 it is evident that huge quantity of treated water (14-31%) is to spent for backwashing.

Table 5.12: Monthly expenditure on O&M in the study plants

Plant Location	Electricity Cost	Manpower cost	Depreciation & Other cost	Total expenditure	Cost of Production Tk/m^3	Cost of Backwashing (Tk/month) (%of O&M)
Sirajgonj	55813.00	42500.00	30583.00	128896.00	2.21	41305/- (32%)
Hobigonj	55910.00	44916.00	30583.00	131409.00	1.91	19558/- (15%)
Gopalgonj	37799.84	50833.00	30583.00	119215.84	3.50	46410/- (39%).

The above table 5.11 shows that about 32, 15 and 39% of the total O&M cost is being spent on backwashing which indicates poor performance and a risk of failure of the system unless appropriate measures are taken.

Overview:

1. The sand used in the filter bed of existing plants are not well graded. The uniformity coefficients are more than the design values(<1.3).
2. The design and placement of wash water trough is found wrong and it is one of the major defects for which uneven and poor washing of the filter beds occurred. Trough should be placed in the middle parallel to the length of the filter and the lateral travel should be limited to 1.06m and vertical distance to 0.6m above the expanded bed.
3. The filters are designed to be backwashed at lower backwash rate than required *(Table 5.6)*.
4. Expansion of filter bed was designed at a very low value (0%) which results insignificant hydrodynamic shear to result effective cleaning.

5. The raw water quality does not qualify the required for direct filtration.
6. The filter runlength is decreasing with time.
7. Incomplete backwash, design loophole and use of raw water quality which does not qualify for direct filtration are the major causes for poor performance of the treatment plants.
8. Nozzle type underdrain system which renders non-jet action can not play appropriate role in effective cleaning. This type of underdrain provides backwash water with lower velocity and subject to frequent clogging of the opennings.
9. The filters are taking as much as 4 times of impurities load that it has ability to handle.
10. The design did not consider the variation of raw water parameter and made *type design* regardless of area and water parameter.
11. Backwash water requirement is 14 to 31% of total water production which seems very high with respect to ideal situation.
12. If the design included flocculation and sedimentation tank it would cost extra 18lakh, however, at present additional recurring cost is being required for backwashing of the beds. It is estimated that with the cumulative additional recurring cost of 4.5, 12 and 4 years one unit of flocculation and sedimentation tank could be installed in Sirajgonj, Hobigonj and Gopalgonj respectively.

CHAPTER-6

Suggested Modifications of The Existing Plants

6.1 General:

It has been clear from the previous discussion that the study plants are suffering from a lot of Design and Operational problems. In this chapter modifications are suggested to solve/minimize these problems in order to improve the performances of the plants.

6.2 Outline of Modification of Existing Plants

The existing plants do not have provision for expansion or very little scope for modifications. Moreover, some problems are inherent. Nevertheless, Keeping in mind the problem of renovation of the existing structure suggestions have been made within the design principal and the facilities available in the sites. It is expected that it will results some improvements.

- **Inlet Water Distribution/Division chamber**:

Raw water is first carried to the division chamber for distribution of raw water into two filters. Thorough mixing will result good oxygenation in the chamber and further oxygenation in subsequent cascade steps. As we discussed in previous sections too much oxygenation results quick forming of sludge blanket on the bed. To minimize oxygenation rate the spout of raw water pipe may be extended to submerge into the water of the division chamber.

- **Cascade:**

Two step cascades of total fall height 1.25m shows optimum aeration. However, it will be difficult to change the height and to reduce the number of cascade as it requires dismantling of concrete structure which may lead to damage to other parts of the plant. So cascades will remain unchanged. However, 75 to 100mm size brick chips (if possible iron precoated) may be placed in the cascade steps. This will adsorb Fe^{2+} and thus reduce burden on filters to some extent.

- **Filter chamber:**

Filter area is designed to filter a maximum of $204m^3/hr$. of water at a filtration of 7.5m/hr. However, plants are being operated at lower rate (5.4 to 6.3 m/h). Since runlength is becoming shorter, a reduction in filtration rate may increase the runlength. But total production of the plant will be reduced further. All the plants are not capable of meet the present water demand of the respective pourashava. It is reported that the plants under present conditions (DPHE Data Book) can meet only 21 to 40% of the total demand. The study considered that production capacity should not be compromised so the filtration rate and the area of bed will be kept unchanged.

- **Terminal Head:**

Since filter runlength is directly proportional to terminal head careful examination has been made if the terminal head can be increased. For existing IRPs there is a free fall(min) of 0.4, 0.55 and 0.8m in Sirajgonj, Hobigonj and Gopalgonj respectively. So, an increase in terminal head is possible. It is estimated that if the terminal head is increased by 0.15, 0.30 and 0.55m the length of run will be increased by 2-3 hours in the IRPs as demonstrated in figure 5.3.

- **Sand grain size:**

Filter grain size 1 to 2mm with uniformity coefficient <1.3 is rather expensive. As a matter of fact choice of grain size is a compromise between filter resistance and effluent water quality. An increase in the size of sand results higher runlength, however leads to an increase in backwash rate. Lower size sand gives better effluent quality. The effluent water quality given in table 5.1 reveals that the quality needs to be improved. *Considering operational and economical point of view the study considers that effective grain size may remain within a limit of 0.8 to 0.9mm with uniformity coefficient < 1.40.* Since no change is proposed for the filter box the *bed thickness is also proposed to remain unchanged.*

- **Filter Cleaning and Back wash rate:**

During study efforts were made to clean the filters effectively, the wash trough was found to be too high to take away the iron flocks and impurities. Impurities were settled before the flocks were reached to the trough. So vigorous shaking by wooden stirrer was made. It was found that better wash was performed (turbidity,15-18) 5 minutes before normal wash time. **It implies that about 20 to 25% of existing backwash water can be saved.** A sample of wooden stirrer is presented in figure 6.2.

Present backwash rate is much below the required rate. It is required to increase the wash rate, and it can be made possible either with the aid of OHT or through installation of a new high capacity pump, about 1000m^3/hr, through which backwash can be made at a rate of minimum 54m/hr. In case OHT is used for backwashing separate bypass line (250 to 300mm diameter) from delivery line to plant has to be constructed. A rough estimate shows that both the options cost almost same – about Tk. 5 to 6 lakh. By trial run and adjustment the wash rate can be fixed. Which option will be adopted depend on the water works authority, however, study considered that two options are feasible and the wash rate should be increased to at least 54 to 55 m/hr.

- **Under drainage System:**

Present underdrainage system can not play desired role in effective cleaning of the filters. Other system like Lateral manifold would have been effective. But since the filter bottoms were constructed by huge Reinforced Concrete structure it will be very difficult to make any change in it. Under these circumstances, the study decided to keep the filter bottom unchanged. However, yearly cleaning during change of filter sand is suggested.

- **Wash Water Trough**:

A complete design of washwater trough is given for a wash rate of 54 m/hr. Figure 6.1 shows the placement and dimension of the suggested trough to be set in the existing plants.

Design data:

- Q = 0.20 m3/sec
- Width = 0.70m
- Height = 0.40 (including 0.05m freeboard)

In case the wash rate are not increased for any reason, the trough size should be as follows:

- Q = 0.13 m3/sec
- Width = 0.60m
- Height = 0.35 (including 0.05m freeboard)

This trough will be placed perpendicular to the existing trough at the centre and above 0.35m of unexpanded bed. For easy installation the trough can be made by 6mm MS plate (preferably nickel-plated). Otherwise concrete made trough may take longer time to be available for operation.

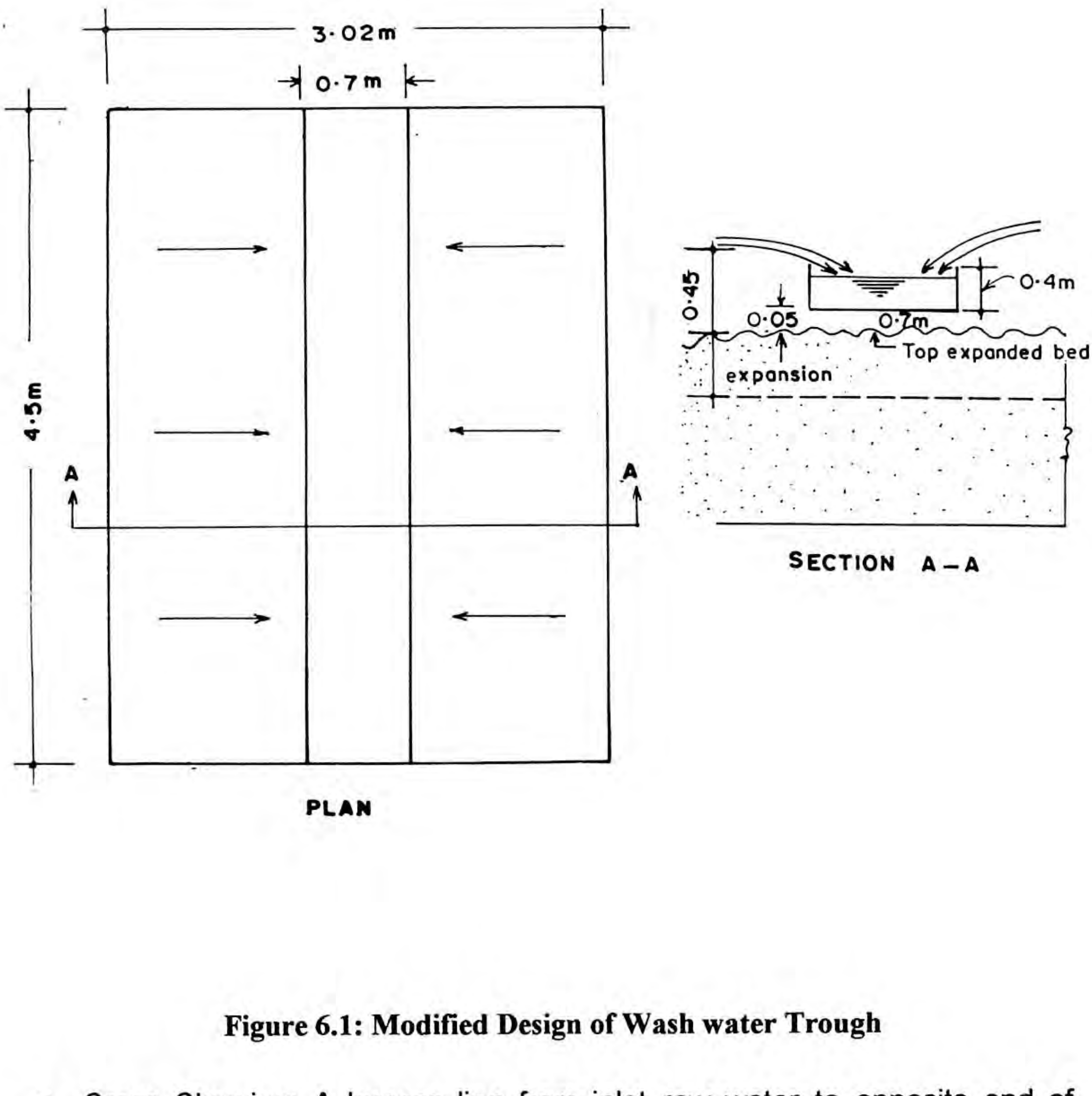


Figure 6.1: Modified Design of Wash water Trough

- Cross Cleaning: A bypass line from inlet raw water to opposite end of trough in filter bed is proposed to construct. The main function of the bypass line will be to flash water on the bed during backwash. It can be done for 3-4 minute. Cross cleaning results better cleaning and reduce backwash time thus save water requirement for backwash.

OPERATION & MAINTENANCE:

⇒ Surface scraping of iron sludge should be done twice a week preferably every alternate day.

⇒ During backwash loosening the sand bed by appropriate stirrer should be done. It will improve the level of cleaning and reduce wash time by 25 to 40 percent.

⇒ In Sirajgonj it is suspected that some filter nozzles are damaged (sand grains were seen in the underdrain of filters). It is suggested to replace some damaged filter and filter nozzles should be cleaned. In general filter nozzles should be washed half-yearly.

⇒ In Hobigonj weekly surface cleaning is done and as such its performance with regard to runlength is better.

⇒ Auto suction cut system be restored in Sirajgonj plant immediately. Otherwise it may damage pumps

⇒ Water level indicator both in CWR and OHT be reinstalled in Sirajgonj and Hobigonj.

⇒ Ceiling Fan with 0.4m shaft on each filter bed should be fixed. It will render good circulation of air and help dissolved foul gases to go away.

⇒ The iron made components properly be painted and overall cleaning of the inside and outside of the plants be emphasized. Aesthetic issue is important in water works to build confidence in consumer's mind.

It is so expected that the overall performance will be improved and result ease of operation. The runlength will be increased particularly in Sirajgonj and Gopalgonj and as a result one time wash will be required. ***In such case percent of backwash water required is expected to decrease to half of present requirement.***

Table 6.1: Expected saving of backwash water after modification

Plant Name	Present Backwash water		Expected backwash water		Saving
	Amount(m3) Per month	Cost(Tk) Per month	Amount(m3) Per month	Cost(Tk) Per month	Per month (Tk.)
Sirajgonj	18690	41,305/-	9,350	20,663/-	20,650/-
Hobigonj	10,260	19,600/-	10,260	19,600/-	-
Gopalgonj	13,260	46,400/-	6,630	23,205/-	23,200/-

Table 6.2: Modified Design Data of different steps of Existing IRPs.

Items	Sirajgonj	Hobigonj	Gopalgonj
Cascade			
• Number of steps	3	3	3
• Fall height	1.65m	1.65m	1.65m
• Oxygen input	7-8 mg/l	7-8 mg/l	7-8 mg/l
Filter			
• Surface area	$13.6m^2$	$13.6m^2$	$9.3m^2$
• Bed height	1.5m	1.5m	1.5m
• Filtration rate	7.5m/h	7.5m/h	7.5m/h
• Filter medium	sand	sand	sand
• Effective Grain size(d_{10})	0.9mm	0.8-0.9mm	0.9mm
• Uc	<1.4	<1.4	<1.4
• Overflow height	1.95m	1.95m	1.95m
• Backwash rate	54m/h	54m/h	54m/h
Chlorination Unit			
• Dosing tank	$1m^3$	$1m^3$	$0.8m^3$
• Solution strength	0.4-2.4%	0.4-2.4%	0.4-2.4%
• Chlorine dose	2-6 mg/l	2-6 mg/l	2-6 mg/l
Clear Water Tank			
• Effective volume	$151m^3$	$151m^3$	$135m^3$
• Residence time	45 min	45 min	48 min
Sludge Tank Volume	$125m^3$	$146m^3$	-
High lift/backwash pump			
• High lift capacity	$204m^3/h$	$204m^3/h$	$190m^3/h$
• Backwash capacity	$>750m^3/h$	$>750m^3/h$	$>750m^3/h$
• Backwash(optional) by OHT	✔	✔	-

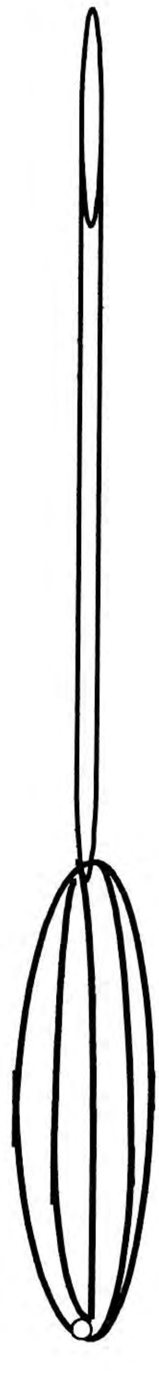


Figure 6.2: Pipe Stirrer for Particle abrasion and shear in the filter bed

CHAPTER-7

Future Design Guidelines

7.1 General:

In this section an outline of design for future treatment plant to be constructed is provided based on the discussion made in chapter-5. Design loophole and reasons for poor performances were highlighted. The intention of this author is to draw attention of the future design so that they can keep the basic problems that have been identified in the study in their mind and also find a guideline for the detailed engineering design. *The detail engineering design has not been provided because it is beyond the scope of this thesis and particular plant is not identified to be designed. Nevertheless some process design is provided in appendix-H considering a same capacity of existing plants.* The future designer can, based on this out line design, perform suitable design of plant according to the need of particular area which can meet the specific field demand/conditions.

7.2 Basic Consideration:

- Selection of unit process should be made on the basis of raw water quality investigation and model test.
- Initial capital cost should not be always given top priority. Operational issues such as O&M cost, ease of operation etc. should be taken into consideration.
- Initial iron concentration when exceeds 2ppm direct filtration should be avoided.

7.3 Laboratory Model Test, Waterquality Investigation & Selection of Unit Process

Detailed water quality investigation was carried out for the raw water of the study plants. The findings of the investigation is given in table 5.10 and Annex-F & G. which reveals that iron concentration is 8.0 and above for Sirajgonj and Gopalgonj. For Hobigonj the concentration is 4.5 ppm.

The turbidity of water on the filter bed is 32 and 43 NTU in Sirajgonj respectively. However in Hobigonj it is 10NTU.

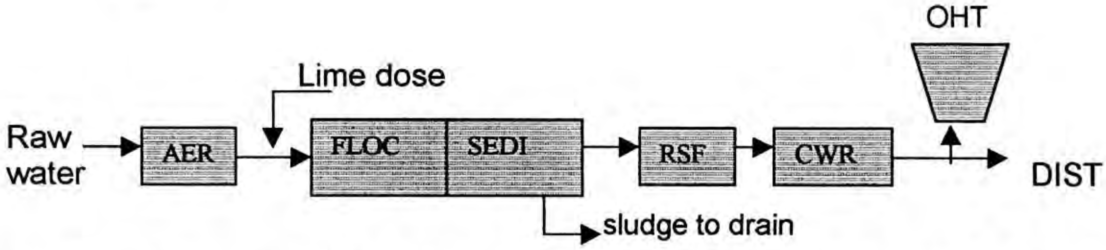
According to AWWA the raw water quality of Sirajgonj and Gopalgonj do not qualify for direct filtration.

A laboratory model test was also carried out with the water samples collected from the study plant in BUET laboratory. Laboratory model test reveals that:

- Settling rate of precipitated micro iron particles are very slow, as a result only plain sedimentation is not going to reduce the suspended particles load on the filter bed. (Annex-F & G.)
- Flocculation and subsequent sedimentation can enhance the settling performance over 25 to 60% for a detention time of 60 minute in Sirajgonj and Gopalgonj respectively.
- In Hobigonj, however, flocculation and subsequent sedimentation can not reduce turbidity load(micro iron particle) even for a detention time of 90 minute.
- Flocculation with NaOH can reduce the turbidity load about 80% for a detention of 15 minutes in Hobigonj.
- Flocculation with lime can reduce the turbidity load about 60% for a detention of 60 minutes in Sirajgonj.

Water quality investigation and laboratory model test thus reveals that pretreatment of the raw water by sedimentation preceded by flocculator is necessary in the treatment process.

Flow Diagram:



RW = Raw Water ; AER = Aeration ; FLOC = Flocculator ; SEDI = Settling basin ; RSF = Rapid Sand Filter ; CWR = Clear Water Reservoir ; OHT = Over Head Tank ; DIST=Distribution Network ;

Figure 7.1: Flow diagram of required Iron Removal Plants

7.4 Possible Layout of IRP

The following components would be required in the IRPs of the study plants for desired level of operation and quality of water:

- A division chamber for collection of raw water from different wells and for equal distribution over the filters.
- Flocculation channel.
- Settling tank.
- Cascades for aeration.
- Single media rapid sand filter with lateral manifold underdrain.
- Pumping Room.
- Post chlorination unit and Overhead Tank (where necessary).

7.5 Design Outlay

7.5.1 Aerator:

Reference is made to table 5.2. In all three plants maximum oxygenation is occurred in 2nd step cascade. Increase of pH after this step is not prominent. So one or two step cascade with height 1.00-1.25m can be selected.

Design Data:

Type

Method	:	Cascade
Material	:	RC structure
Water Application	:	gravity flow from V-notch

Unit

Number of step	:	2 steps
Fall height	:	1.25m (total)

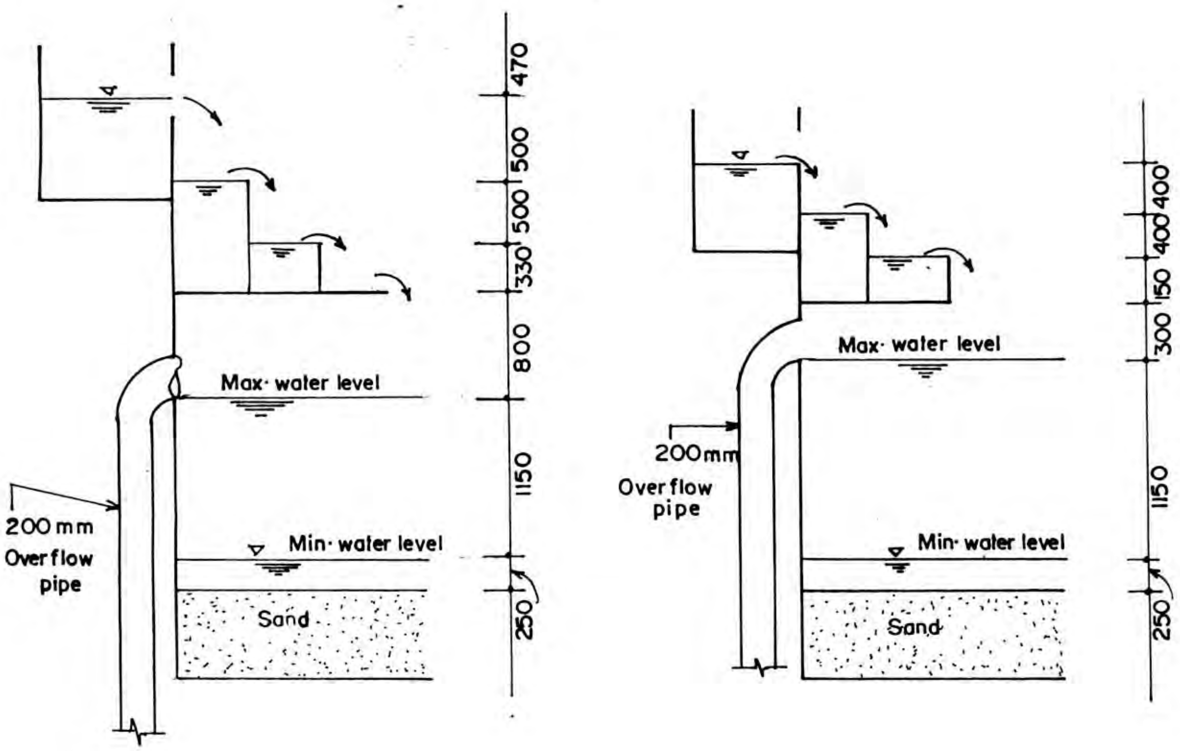


Figure 7.2: Existing and Proposed design of Cascade of IRPs

7.5.2 Flocculator:

Based on the model test detail flocculator design is made. An out line of flocculator design is presented below for Q=204 m3/hr. (case Sirajgonj)

Type

Method	:	Baffled channel Flocculation (gradually tapered)
Direction of flow	:	Horizontal
Material	:	RC concrete

Unit

Number of Channel	:	3 nos. (parallel)
Height of Channel	:	1.0 m

Other parameter	Channel-I	Channel-II	Channel-II
Length of channel	23.4m	15.25m	25m
Width of channel	0.91m	1m	1m
Number of baffle	52	23	27
Spacing of baffle	0.46m	0.68m	1.06m
Length of baffle wall	0.6m	0.45m	0.28m
Free board	0.20m	0.20m	0.20m
Headloss	0.20m	0.05m	0.03m
Mean velocity Gradient	85-95 sec^{-1}	50-55 sec^{-1}	25-35 sec^{-1}
Camp number Gt	25500-28500	9000-9900	7500-10500
Floor slope	0.45%	0.5%	0%

7.5.3 Sedimentation Unit:

Rectangular tube settler has been proposed for its compact size and efficient performance.

Design criteria:

Laminar flow will prevail

Reynold number Re<200(max 500)

SOR (S_o) = 21.6m3/day/m2

Plate length a = 0.5 to 1m

Interplate distance = 0.025 to 0.1m

Scale up factor = 1.65

Detention time $\cong$ 10min

Design:

Type	:	Tube settler
Flow direction	:	Upflow
Tilting angle α	:	60^o
Surface area A	:	11.45 m^2
Detention time	:	10.35 min (corrected by scale up factor)
No. of tubes	:	90
Length of each tube	:	2.3m
Vertical height H	:	2m
Width of a tube w	:	0.05m
Thickness of tube	:	0.005m

The tubes are placed in three layers with 30tubes in each layer.
Satisfy for Re = 170 (<200) ; Froud no. Fr = $15x10^{-5}$

$$\text{Dtention time } t = \frac{H/\sin\alpha}{v_O} = 380\text{second or } 6.3\text{min.};$$

after correction by scaleupfacor1.65 t = 10min.

7.5.4 Filter bed:

Previous designed filtration rate of 7.5m/hr. is selected. Empirical formulae give the following filter dimensions:

Type of Filtration	:	Rapid sand filtration
No of filter unit(n)	:	2 no.
Area of each filter unit(a)	:	13.5 m2
Filtration rate (v)	:	7.5 m/hr.
Grain size	:	0.8-0.9mm (U_c<1.4)
Depth of bed	:	1-1.25m

7.5.5 Washwater Trough:

The washwater trough will be placed in the middle parallel to the long direction of the filter bed so that maximum travel made by the waste water does not exceed 1.06m. In table 5.6 it was shown that for grain size 1.0mm the required wash rate is about 52m/hr. Detail design is made which not only fulfill the need of wash flow requirement but also comply the standard AWWA practice. The summery of the design dimensions are given below :

A rectangular section trough is selected.

Type	:	Rectangular
Width (b)	:	0.70m
Height (h_1)	:	0.40m (including .05m freeboard).

The trough should be placed at 0.35m above from the unexpanded bed. (0.3m for 20% expansion and 0.05m for min. clear gap).

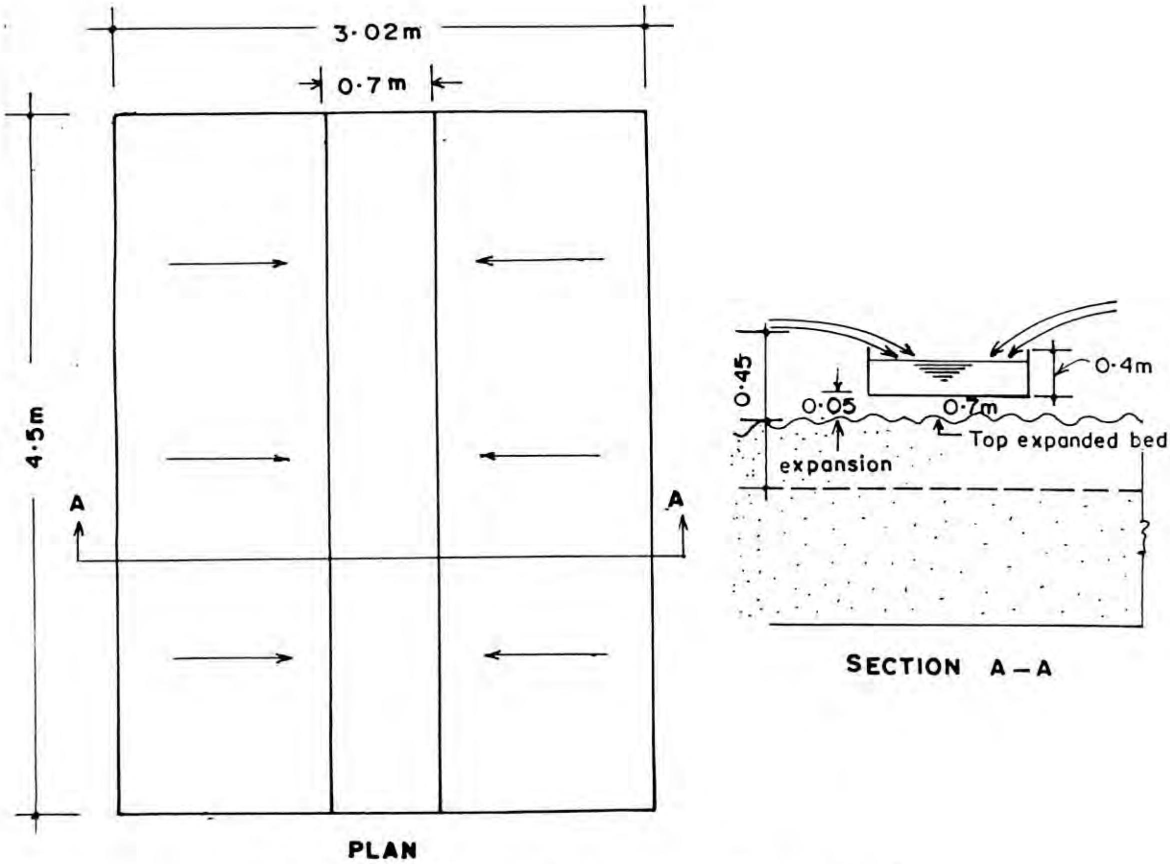


Figure 7.3: Proposed Washwater Trough Design

7.5.6 Filter Under drain:

On the basis of discussion made in chapter-5 Manifold lateral pipe under drainage system is selected. The principal issue in designing underdrainage system is to provide uniform distribution of backwash water. The basic approach is to create a pressure drop through the perforations on the in the underdrain system which is considerably greater that the changes in pressure that occur in the underdrain piping due to friction loss, velocity head loss etc.
A number of thumb rules have been successfully been used by the designer to design manifold and lateral to accommodate wash rates from 9 to 55 m/hr.

Dia of orifice = 6 to 12mm
Spacing of orifice 75 to 300mm
Spacing of laterals = 75 to 300mm
Ratio of cross sectional area of manifold to sum of cross sectional area of laterals = 1.75 to 2.0
Ratio of sum of area of orifice to total filter area = 0.0015 to 0.005
Ratio of lateral length to its diameter = <60

$$\frac{v^2}{2g} = \frac{1}{6}\frac{v^2}{n^2 D_0^4}$$

=1 to 4

with few trials following underdrain is finalized that satisfy the common practice and rules.

Design data:

underdrain for washrate v = 54m/hr.

a) Manifold

Diameter	:	375 mm
Material	:	6 mm thick MS pipe
Length	:	4.35m (150 mm for clear space)

b) Laterals

Material	:	PVC
No. of lateral	:	50 no.
Length	:	1.2m
Spacing	:	175 mm
Diameter	:	38 mm
No. of orifice	:	37per m^2 of bed
Spacing(a pair of orifice)		240 mm

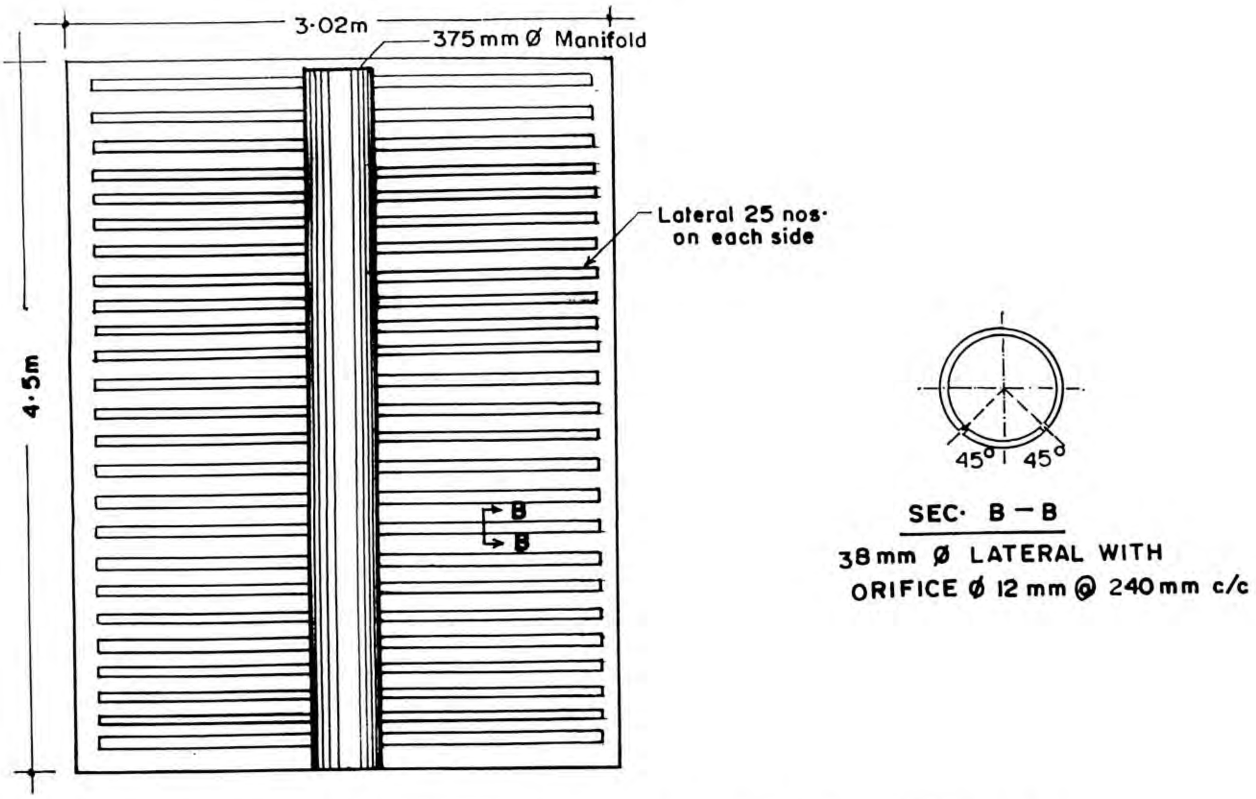


Figure 7.4: Proposed Filter Bottom Lateral-Manifold System

The manifold lateral system needs gravel support to prevent filtering materials from entering and blocking the underdrain. Two conditions have to be satisfied:

1. The top gravel layer should be fine enough to prevent filtering materials from entering and clogging the opening between the gravel grains. It also should not be expanded.
2. The bottom layer should be so coarse that it cannot be dislodged by the jet emerging from the orifice.

Design criteria :

- Lower grain size limit of top gravel layer is between 4 to 4.5 times the diameter of filtering materials.
- Ratio of upper gravel size limit of gravel below to the lower gain size limit of gravel above should be below 4.
- Lower grain size limit of bottom gravel should be 2 to 3 times the orifice diameter.
- The thickness of each layer $\geq$ 0.07m
- Head loss $\approx$ 0.4m for 54m/hr. wash rate.

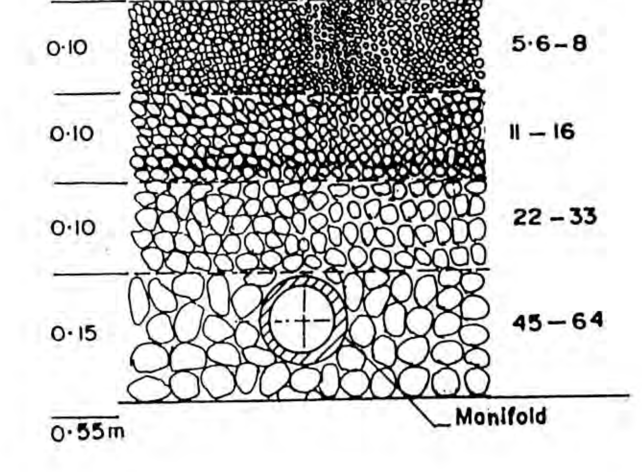


Figure 7.5: Gravel Support Filter underdrain

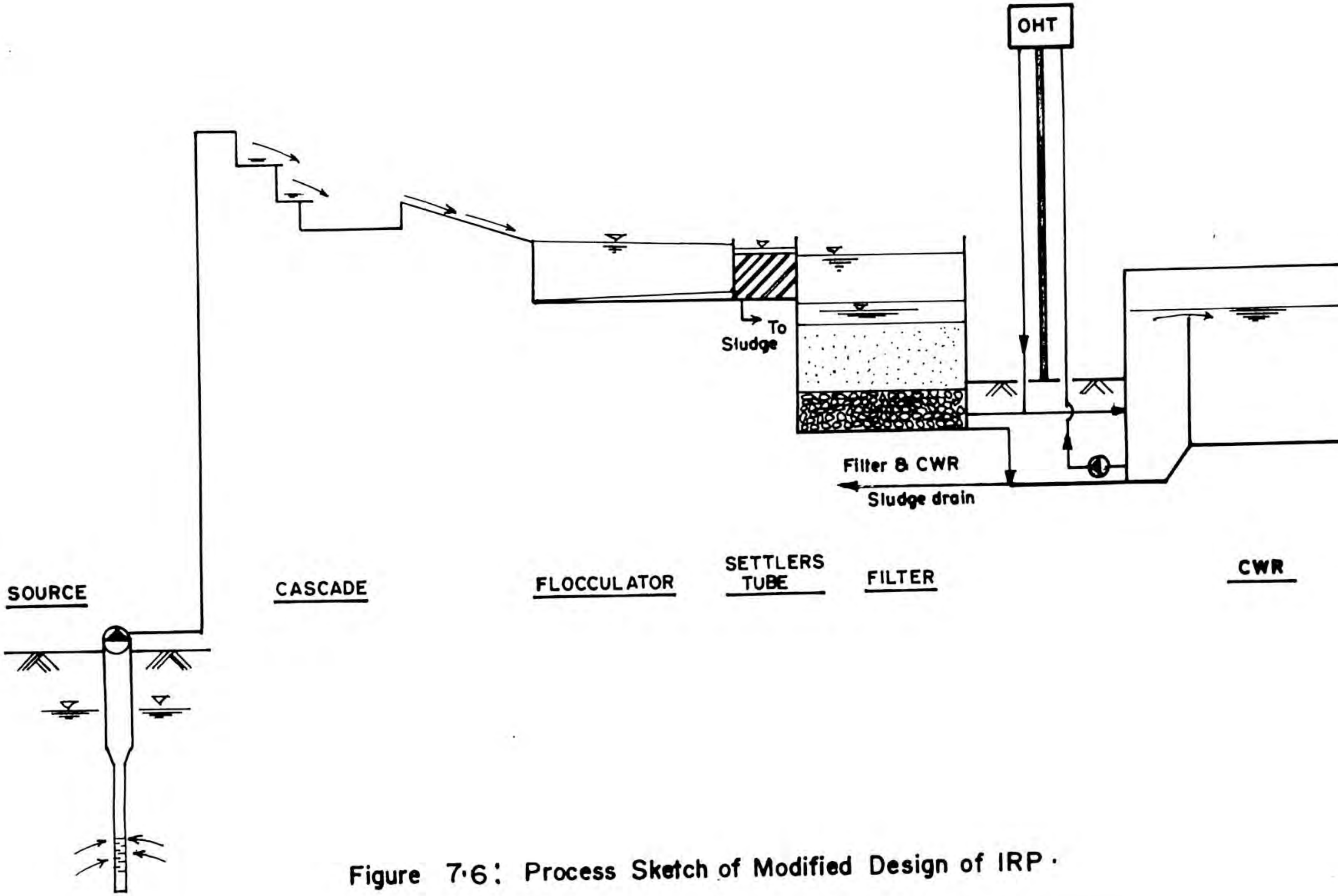


Figure 7·6: Process Sketch of Modified Design of IRP ·

CHAPTER-8

Economic Analysis of Existing and Proposed IRPs

8.1 Additional Cost of O&M

Various researchers showed, on the basis of their experiment, that backwashing in direct filtration with water alone should not exceed 4-6% of total production. AWWA also recommended limiting the requirement of wash water to a maximum of 6%. In the study plants it was found that washwater requirement is abnormally high. The reasons for this high requirement of backwshing are described in previous sections. This chapter will try to describe the cost implication of the problem related to backwashing.

Table 8.1: Additional Cost of Operation & Maintenance

Name of the Town	Water production Per month (m3)	Actual Backwash Amount / Month (m^3)	Ideal Backwash amount(m^3)/ month	Excess Backwash amount(m^3)/ Month (3-4)	Cost of excess Backwashwater (Tk.)/month
1	2	3	4	5	6
Sirajgonj	77600	18700	4656	14044	31037.25
Hobigonj	77026	10240	4622	5618	10786.50
Gopalgonj	45180	13260	2710	10550	36925.00

Table 5.10 shows the monthly cost incurred for backwashing in excess of that in standard practice. The costs are calculated considering 6% backwash water as a standard limit. The annual additional cost then stands at Tk. 3.72, 1.29 and 4.43 lakh respectively.

If the plant would design with conventional process i.e. aeration, flocculation, sedimentation and filtration process to make the raw water qualified for filtration then the plant initial cost would have been increased by 15%. The total cost of construction of a plant was about 125 lakh on average. So Tk.18 lakh more would be required should there was a flocculation sedimentation unit with the plant process.

The present design might have saved this money. However, excess cost for backwashing at the rate of Tk.4.94 lakh per year, in case of Sirajgonj, for example, has been consumed the saving of the initial expenditure in 4.5 years. Similarly in Hobigonj and Gopalgonj saving money is realizing in about 14 and 4.0 years respectively.

If it is a fact that the designers had designed the plants as such to save initial investment then it can be stated that they could not save the money rather some operational problem have been shouldered on the beneficiary pourashava. The pourashava with lack of technical capabilities are in great problem with these problems.

The study attempted to provide a design out line in order to make the plant process economical with a special attention to controll activities which incurred such additional costs e.g. backwash amount. Measures for effective cleaning of the filter beds are taken. Some of them are to construct well-performed underdrain, well designed wash water trough and to clean the beds by OHT etc.

A cost comparison is given in table 8.2 where it is shown the per cubic meter water production cost between existing and proposed IRPs.

The detail of the calculations are given in Appendix-B & H. The table 8.2 reveals that the modified design will reduce the production costs by 30, 20 and 26% in Sirajgonj, Hobigonj and Gopalgonj respectively.

Table 8.2: Comparison of Production Costs

IRP Town	Existing Plant						Modified design						Production cost (Tk.)	
	Electricity	Manpower	Depreciation	O&M	Total recurring cost	Net production	Electricity	Manpower	Depreciation	O&M	Total recurring cost(Tk.)	Net production (lac m^3)	Existing	Proposed
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Sirajgonj	6.71	5.10	0.6	3.07	15.48	6.99	6.54	5.10	0.60	2.10	14.34	9.20	2.21	1.55
Hobigonj	6.70	5.39	0.6	3.07	15.76	8.20	5.62	5.33	0.60	2.10	13.65	8.86	1.92	1.54
Gopalgonj	4.58	6.10	0.6	3.07	14.36	4.15	4.37	6.10	1.10	2.10	13.67	5.09	3.50	2.60

CHAPTER-9

Conclusions and Recommendations

9.1 Conclusions

Based on the Performance analysis, results and discussions laid down in the previous chapters, following conclusions are drawn :

- It seems from the discussions in the previous chapters that the selection of unit processes of treatment was not made on the basis of pilot plant results or water quality investigation or model test results. Prototype design was made ignoring the variation of water quality in the areas under study.
- Cascade height could be reduced in order to optimize oxygenation.
- The filters are designed to be backwashed at much lower backwash rate than required which results incomplete backwash.
- The design and placement of wash water trough is found wrong.
- Expansion of filter bed was designed at a very low value (0%) which results insignificant hydrodynamic shear to result effective cleaning.
- The raw water quality does not qualify for direct filtration.
- Low rate of backwash, wrong design of wash water trough, use of strainer type filter underdrain and use of raw water quality which does not qualify for direct filtration are the major causes for poor performance of the treatment plants. The poor performances are exhibited through short runlength, increase of O&M cost, decrease of effluent water quality etc.
- Model test reveals that pretreatment through flocculation and sedimentation was necessary for treatment of high concentration of iron of 4.5 to 8.7ppm.

- Observation and experience reveals that a maximum limit of 2ppm iron can be allowed for direct filtration. Beyond this limit pretreatment process must be included in the treatment process.

- Proposed modification as suggested in this study can reduce the backwash water requirement to a half.

- The modified design is expected to reduce the production cost by 30,20 and 26% of present cost incurred by existing IRPs

9.2 Recommendations

Following recommendation are hereby made based on the findings of the study:

- Pre-design investigations or model tests are to be performed before such plants are installed in large scale.

- Detailed and careful investigation of raw water quality should be carried out in order to select appropriate unit processes.

- Backwashing by Air-water scouring should be introduced in the future plant design.

- Provision of cross cleaning should be kept in Filter unit.

- Disinfecting by chlorine or bleaching powder should resume.

- A mini laboratory should be set up in each of the Treatment Plant complex; else testing should be done on regular basis from nearby DPHE laboratory.

- Pourashava authority should pay more attention to the water supply system including the IRP. Frequent visits may be made to the treatment plant for close monitoring of the plant activities and to ensure desired level of operation and maintenance.

REFERENCES

1. Ahmed, F and Ahmed M. F, 1981, "*Development of community type IRP for water supply in rural Bangladesh*", published by Bangladesh Journal of Water resource Research, vol. 4 No.1, Dhaka, Bangladesh.

2. Azim, Z, 1991, "*Study of the performance of Existing Iron Removal Plants Developed by Unicef*", A M.Sc. Engineering thesis, Department of Civil Engineering, BUET, Dhaka, Bangladesh.

3. Adekoya, O.A, 1995 "*Optimization of Iron Removal in an Aeration/Filtration Process*" A M.Sc. Thesis, EE 179, IHE, Delft, The Netherlands.

4. AWWA, 1990, American Water Work Association publication, "*Water Quality and Treatment*", 4th. edition, Mcgraw-Hill Book company, New York., USA, 1990.

5. AWWA, 1951 "*Water quality and treatment*", A manual published, New York, USA.

6. Ahmed, F, 1987 "*Low cost iron removal for hand pump tubewells*" Ph.D. thesis.

7. Amirtharaja, A. 1978 "Optimum Backwash of Sand Filters", J Environmental Engineering Division, American Society of Civil Engineers, USA.

8. Baylis et al. 1971 "*Filtration" in: Water Quality and Treatment,* A hand book of Public Water Suppl;ies, Third edition Mc.Graw Hill book, New York.

9. Clark, J. W, Viessman, W. Jr. and Hammer, M. J. 1971 "*Water supply and pollution control*", 2^{nd} . edition, published by International Textbook Company.

10. Committee Report, *The Status of Direct Filtration*", J.AWWA Vol.72, 1980.

11. Degremont, 1991, "*The Water Treatment Handbook*", H.K.Elliott, London.

12. DHV, 1992 "*Evaluation of performance of Iron Removal Plants*".

13. DPHE Water Supply & Sanitation Report 1991, "*Operation and Maintenance Mannual for IRP-Gopalgonj*" Program office DPHE-Dutch Water supply and Sanitation project, Dhaka.

14. DPHE, 1998 "*Data Book for Urban Water Supply and Sanitation*" A DPHE publication, Dhaka.

15. Eckenfelder, W.W, "*Water Quality Engineering for Practicing Engineers*", published by Barnes and Noble, Inc., New York, printed in USA.

16. Ehlers, Victor, M., Steel Ernest, W, "*Municipal and Rural Sanitation*" Tata McGraw Hill publishing company Ltd. New Delhi, Sixth edition, 1976, page 24 to 73.

17. Fair, G. M., Geyer J. C, and Okun, D.A 1968 "*Water and Waste Water Engineering*, Volume-2, John Wiley & Sons Inc. NewYork.

18. Ghosh, M..N; O' Conner J. T. and Engelbrecht, R. S. 1966 *Precipitation of iron in aerated ground water*".

19. Hammer, M. J, "*Water and Waste water technology*", (SI version) John Wiley and sons, Inc. page- 249 to 251.

20. Huisman, J, 1986 "*Rapid Filtration*", published by International Institute for Infrastructural, Hydraulic and Environmental Engineering, The Netherlands.

21. Ibrahim, A.K.M, 1997 "*The Development of Iron Oxide Coating on Filter Media and its Role in Iron Removal*" A M.Sc. Thesis, SEE 019, IHE, Delft, The Netherlands.

22. Ives, K. J. 1969, "*Theory of Filtration*" International Water Supply Congress and Exhibition, Viena.

23. King, H. W, Wisler, O, J.G.Woodburn, *"Hydraulics"*, published by Jhon Willy & Sons Inc. New York, 1948.

24. Mintz, D. M., 1951 *"Kinetics of Filtration"* Doki Acad, Nauk. USSR.

25. Met calf and Eddy, "*Wastewater Engineering, Treatment, Disposal, Reuse*", McGraw Hill Book Company, New York, USA, page 180 to 310.

26. O&M Working Group Report, 1996, "*Upgrading Drinking Water Treatment Plants A Mannual for Design and Operation* " Water Supply and Sanitation Collaborative Council, WHO, Geneva.

27. Paul, S. "*Ground water and well*". A reference book for the water well industry, published by Johnson Division, VOP Inc., Page 345 to 362.

28. Popel, H.J. and Post, P.J.H. 1994 "*Aeration and Gas Transfer*" published by International Institute for Infrastructural, Hydraulic and Environmental Engineering, The Netherlands.

29. Robinson, Jr, L.R. and Dixon, R.I. 1968 "*Iron and Mangares Precipitation in low alkalinity ground water*" water and sewage works.

30. Stumm, W. and Lee, G,F, 1961 "*Oxygenation of Ferrous Iron*", Industrial and Engineering Chemistry.

31. Sung, W and Forbes, E., 1984 "Some considerations on iron removal" Journal of Environmental Engineering, A.S.C.E.

32. Sung, W and Morgan, J. J. 1980, "*Kinetics and product and Ferrous iron oxygenation in aqueous system*"

33. Weber, W. J Jr. 1972 "*Physicochemical Process for Water Quality Control*" Jhon Wiley & Sons, Inc. USA

Appendix-A

Calculation of Amount of Backwash Water

Sirajgonj:

Source	Time (min)	Water height at CWR		Depth (m)	Surface Area (m^2)	Water Vol (m^3)
		Before BW(m)	After BW (m)			
CWR	Filter-1 11.30-11.52am 22min	3.78	0.60	3.18	40.00	127.20
IRP	22min	-	-	-	80*22/60	29.33
						156.53
CWR	Filter-2 1:00-1:20pm 20min	3.70	0.50	3.20	40.00	128.00
IRP	20min	-	-	-	80*20/60	26.67
						154.67

Q_p= 155.60*60/20 = 467m^3/h

Hobigonj

Source	Time (min)	Water height at CWR		Depth (m)	Surface Area (m^2)	Water Vol (m^3)
		Before BW(m)	After BW (m)			
CWR	Filter-1 7:30-7:50 pm 20min	3.70	0.85	2.85	40.00	144.15
IRP	20min	-	-	-	80*20/60	26.67
						170.82
CWR	Filter-2 9:50-10:10 pm 20min	3.74	0.90	2.84	40.58	143.64
IRP	20min	-	-	-	80*20/60	26.67
						170.31

Q_p= 170.56*60/20 = 511.68m^3/h

Gopalgonj

Source	Time (min)	Water height at CWR		Depth (m)	Surface Area (m^2)	Water Vol (m^3)
		Before BW(m)	After BW (m)			
CWR IRP	Filter-1 12:20-12:40pm 20min 20min	3.50 -	0.85 -	2.65 -	35.71 50.22*20/60	94.63 16.74
						111.37
CWR IRP	Filter-2 9:50-10:10 pm 20min 20min	3.35 -	0.75 -	2.60 -	35.71 50.22*20/60	92.84 16.74
						109.58

Q_p= 110.48*60/20 = 331.43m^3/h

Sirajgonj

From the table above it is seen that for Sirajgonj the backwash water spent for one filter is 467.00*40/60=311.33 m^3. So for two filters the amount is 622.66 m^3. In Sirajgonj the production wells continues production for 16 hours (40min stop for each filter bed is considered). The filters are cleaned twice a day however they are not cleaned at the same time. When one filter is under backwashing the other continues for production. So the amount of backwash water will be the amount of water from the CWR plus the amount of water treated by the 2nd filter during the time of backwashing of 1st filter.

Total production of water in a day = 80 m^3/h * 17*2 = 2720 m^3
(5:30 am – 10:30 pm)

each filter stops for 40 min. for backwashing, so the amount to be deducted from production is 80*40*2/60 = 106.67 m^3

So, net production = 2613.33 m^3

So, Backwash water required = 622.66*100/2613.33
= 24% of treated water.

Hobigonj:

In this town three PWs are running at a time, one with a capacity of 80m3/hr. and the rest two with 38.63 m3/hr each.

Total production of water in a day = 80 m^3/h * 16.66 + 77.26*16.66 = 2619.95 m^3
(5:30 am – 10:10 pm)

water spent for cleaning a filter is 512*20/60 = 170.67 m^3
For two filter it is 170.67*2 = 341.33 m^3.

each Filter stops for 20 min. for backwashing, so the amount to be deducted from production is 78.63*20*2/60 = 52.42 m^3

So, net production = 2567.53 m^3

So, % of Backwash water required = 341.33*100/2567.53
= 14% of treated water.

Gopalgonj:

In this plant the water inflow in each filter is 50.22m^3/hr..

Total production of water in a day = 50.22 m^3/h *2* 15hr. = 1506.60 m^3
(5:30 am – 10:10 pm)

water spent for cleaning a filter is 331.43*40/60 = 221.00 m^3
For two filter it is 221.00*2 = 442.00 m^3.

each filter stops for 40 min. for backwashing, so the amount to be deducted from production is 50.22*40*2/60 = 66.96 m^3

So, net production = 1439.64 m^3

So, % of Backwash water required = 442.00*100/1439.64
= 31% of treated water.

Appendix-B

Operation & Maintenance cost calculation of an IRP

In calculating the cost of O&M three major costs are considered, they are a)costs of Electricity, b)costs of Manpower and c)costs of maintenance. Since the IRPs are similar in type, capacity etc. The O&M costs are close to each other, however electrical cost varies as it depends on length of pump operation which in turn depends on supply hour, backwash time etc.

a) Electricity cost: The following emperical formula is used to calulate energy consumption of pumps. In this calculation the commercial rate of Tk.3.10 per Kwh unit of electricity is taken.

$$P(W) = \frac{2.78 * H * Q}{1000 * \eta}$$

Where, η efficiency (considered 65%)
Q discharge in m^3/sec
H head in m

Hobigonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk.3.10 (Tk.)
Q=38.63	13	37.50	6.19	80.47	2414.10	7483.71
38.63	13	37.50	6.19	80.47	2414.10	7483.71
80.00	13	37.50	12.83	166.80	5004.00	15512.40
Q=160(to OHT)	7.50	26	17.80	133.50	4005	12415.50
Q=512 (backwash pump)	0.667	10	17.10	11.40	342.00	10260.00
						53155.32

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.53,155.32*1.05*12=Tk.6,69,757.00 .

Sirajgonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk.3.10 (Tk.)
Q=80 80	16 16	37.50 37.50	12.83 12.83	205.28 205.28	6158.40 6158.40	19091.04 19091.04
Q=160(to OHT)	7.50	26	17.80	133.50	4005	12415.50
Q=512 (backwash pump)	1.667	10	17.10	28.5	855	2650.50
						53248.08

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.53,248.08*1.05*12=Tk.6,70,925.80.

Gopalgonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk.3.10 (Tk.)
Q=50.22 (to aerator)	15.00	37.50	16.11	241.66	7249.78	22474.33
Q=148 (to Dist)	7.50	26	16.45	131.66	3949.82	12244.45
Q=335 (backwash pump)	1.33	10	13.77	18.31	549.50	1703.21
						36,422.00

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.36,422.00*1.05*12=Tk.4,58,917.20.

b) Manpower:

The water supply set up of Sirajgonj is as follows:

Sl. No.	Name of the post	No. of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Pumpdriver	4	(1875-3605/-)	2.20
3.	Lineman	3	(1500/-2400/-)	0.90
4.	Bill clerk	1	(2550-5505/-)	0.70
5.	MLSS	1	(900-1530/-)	0.30
				5.10

Hobigonj

Sl. No.	Name of the post	No. of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Pumpdriver	4	(1875-3605/-)	2.00
3.	Lineman	3	(1500/-2400/-)	1.27
4.	Bill clerk	1	(2550-5505/-)	0.70
5.	MLSS	1	(900-1530/-)	0.30
				5.39

Gopalgonj:

Sl. No.	Name of the post	No of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Account Assistant	1	(2550-5505/-)	0.70
3.	Pumpdriver	5	(1875-3605/-)	2.50
4.	Electrician	1	(1500-2400/-)	0.45
5.	Lineman	1	(1500/-2400/-)	0.45
6.	Bill clerk	1	(2550-5505/-)	0.70
7.	MLSS	1	(900-1530/-)	0.30
				6.10

c) Depreciation cost of pumps

considering depreciation rate 10%/year (10year design life)
6,00,000x0.10 =**Tk.60,000**

d)Other

i) Maintenance

The yearly maintenance cost are assumed to be : 3% of investment cost for pipe line, mechanical and electrical parts and 1% of investment cost for civil works. It is also assumed that the cost for civil and E&M cost are 65% and 35% of total investment cost of the system respectively.

The investment cost of the system was 125.00 lakh Taka.

Share for the civil works is 125*0.65=81.25 lakh taka and that for E&M works is 43.75 lakh taka.

So, Yearly Maintenance cost will be

1) for civil works 81.25*0.01=0.8125 lakh taka.
2) for E&M works 43.75*0.03=1.31 lakh taka.

Total = 2.12 lakh taka

ii)Cost of replacing sand (once a year) :

amount of sand for 2 filter = 2*13.5*1.5 = 40.80cum

so yearly expenditure for replacement of sand @ Tk.2328/- = **Tk.94982/-**

Total cost others: 2.12+0.95 = Tk. 3.07lakh

Cost of Production:

i)Sirajgonj:

Total yearly cost of Operation and Maintenance: (a+b+c+d)=6.71+5.1+0.60+3.07
=Tk.15.48 lakh

Total water production: 2720 m^3/day[*ref :appendix-A*]
Total monthly production: 2720m^3/day*30day = 81,600m^3
Total monthly backwash amount : 776.67*30=23300m^3
Net monthly production: 81600-23300=58300m^3
So production cost comes to : 15.48/12 lakh Tk./58300m^3=Tk.2.21/m^3

ii)Hobigonj:

Total yearly cost of Operation and Maintenance:(a+b+c+d)=6.70+5.39+0.60+3.07
=Tk.15.76 lakh

Total water production: 2619.95m^3/day [*ref :appendix-A*]
Total monthly production: 2619.95m^3/day*30day=78598.50m^3
Total monthly backwash amount : 341.33*30=10240m^3
Net monthly production: 78598.50-10240=68358.50m^3
So production cost comes to : 15.76/12 lakh Taka/68358.50m^3=Tk.1.92/m^3

iii)Gopalgonj:

Total yearly cost of Operation and Maintenance:(a+b+c+d)=4.58+6.10+0.60+3.07
=Tk.14.36 lakh

Total water production: 1506 m^3/day [*ref:appendix-A*]
Total monthly production: 1506 m^3/day*30day=45,180 m^3
Total monthly backwash amount : 351*30=10530m^3
Net monthly production: 45180-10530=34650m^3
So production cost comes to : 14.36/12 lakh Taka/34650m^3=Tk.3.50/m^3.

Appendix-C

Calculation of Backwash Rate

The backwash rate for the study plants were calculated from the following equations: (derived from Carman-Kozney equation)

$$v^{1.2} = \frac{g}{130\upsilon^{0.8}} X \frac{\rho_f - \rho_w}{\rho_w} X \frac{p^3_e}{(1-P_e)^{0.8}} X d^{1.8}$$

Where,

$$\upsilon = \frac{497x10^{-6}}{(t+42.5)^{1.5}} \quad \text{and } P_e = \frac{P+E}{1+E}$$

Where, v = wash rate in m/hr.; ν = kinematic viscosity in m^2/hr.

Sirajgonj:

t = 24°C; E = 20%(required) & 1.01%actual; P=0.38 g = 9.81m/sec^2; d=1.2mm; ρ_f/ρ_w= 2.6 for sand

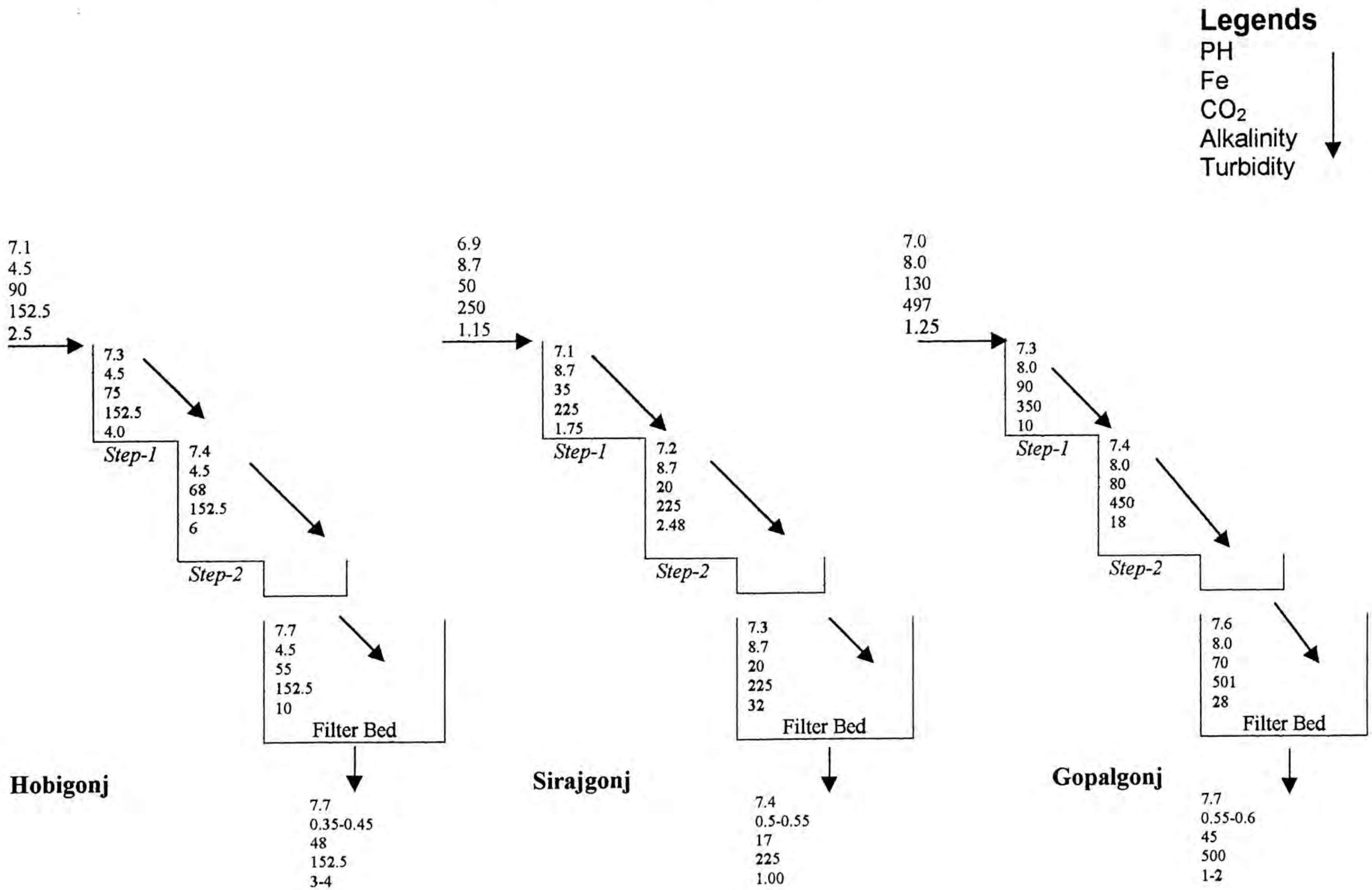
Placing the data in the above equations the required wash is found out 51.6m/hr. say 52m/hr.

Hobigonj & Gopalgonj:

Similarly for Hobigonj and Gopalgonj wash rate 'v' is found to be 52.1 and 51.60 m/hr respectively.

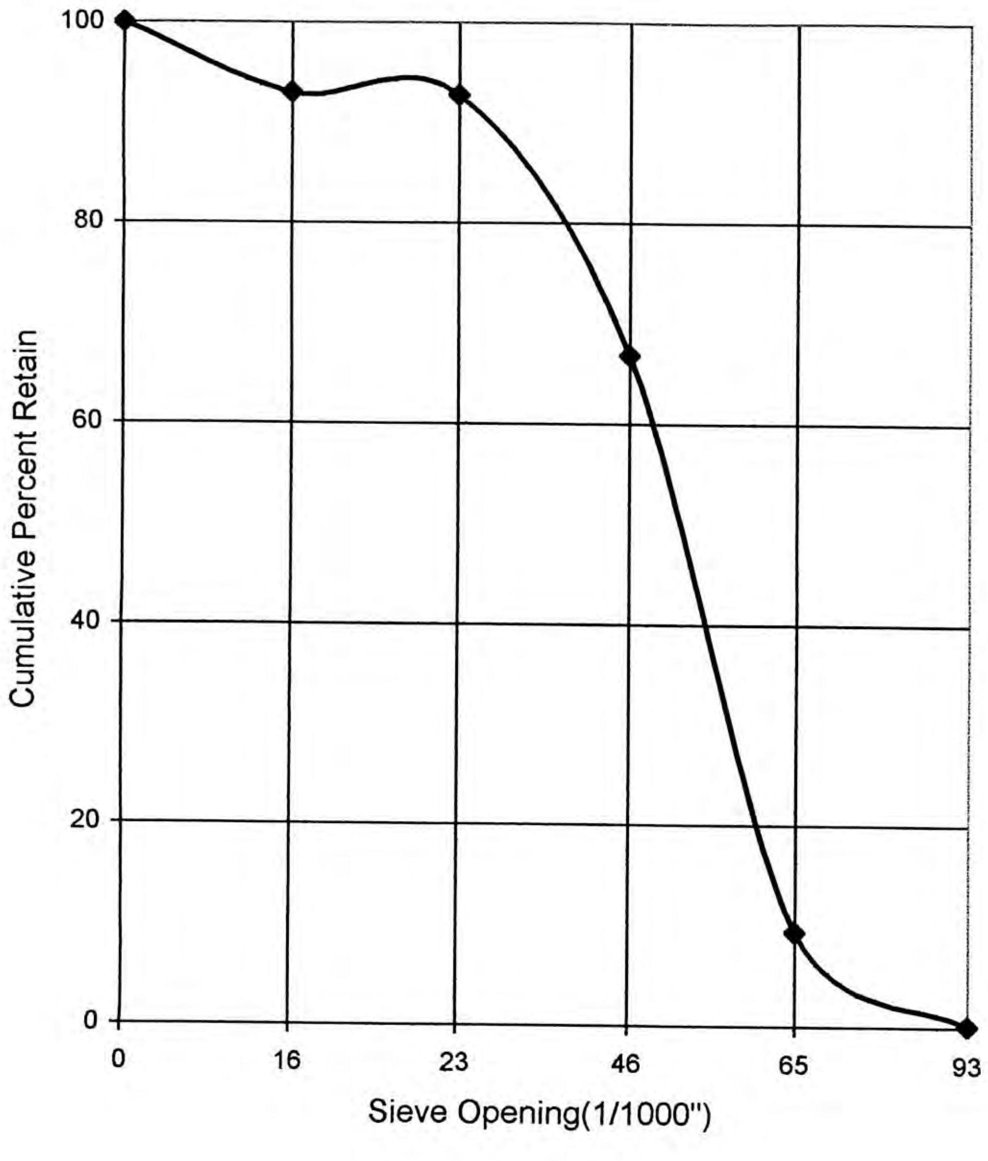
The designed rate should be in between 54 to 60 m/hr.

Water Quality at Different Units



Appendix-E

Sieve Analysis of Filter Materials of Sirajgonj Treatment Plant



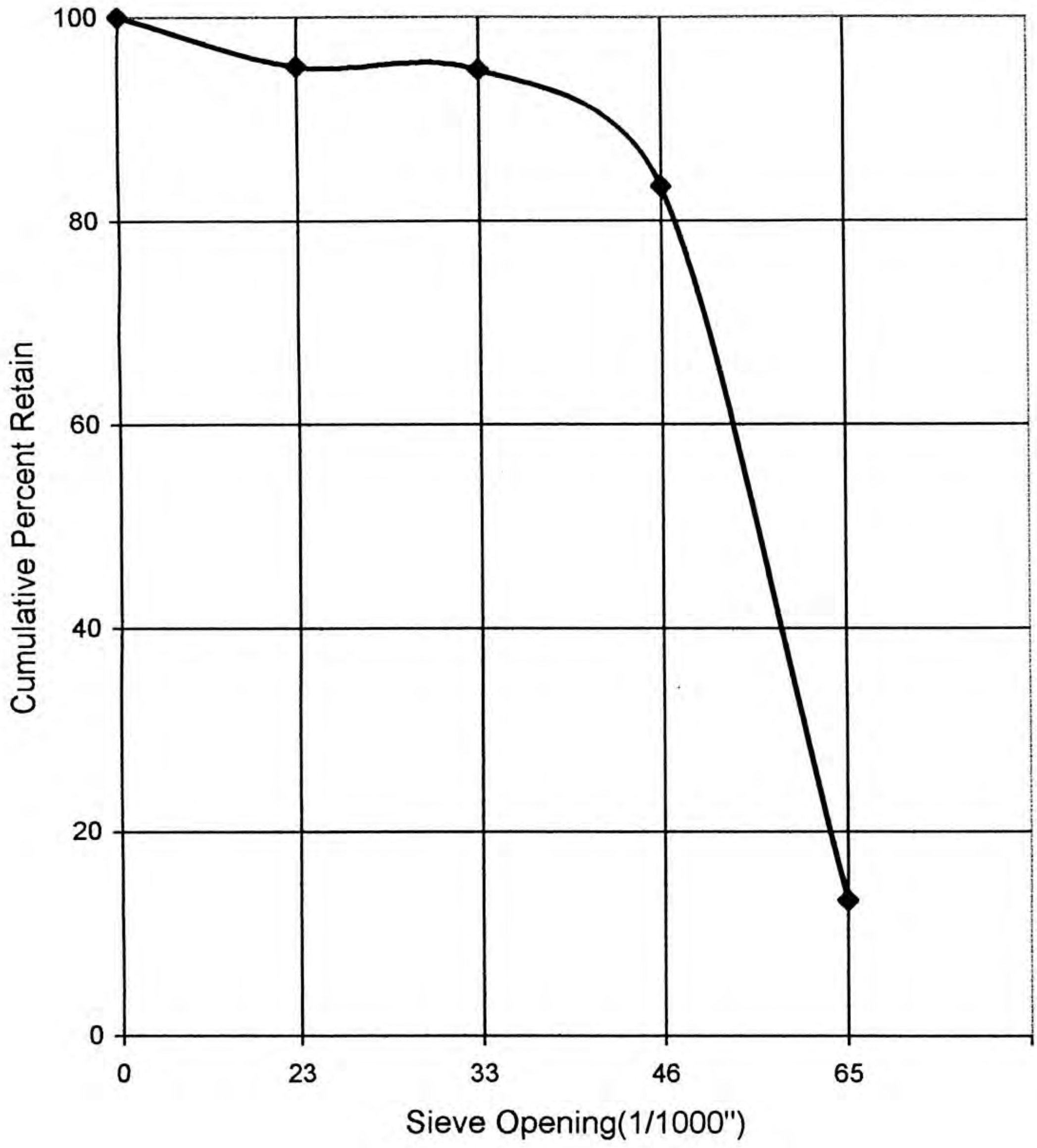
D_{10} = 24/1000" = 0.6 mm

D_{60} = 54/1000" = 1.35mm

U_c = 2.25

Figure E-1: Sieve Analysis of Filtering Materials, Siarajgonj

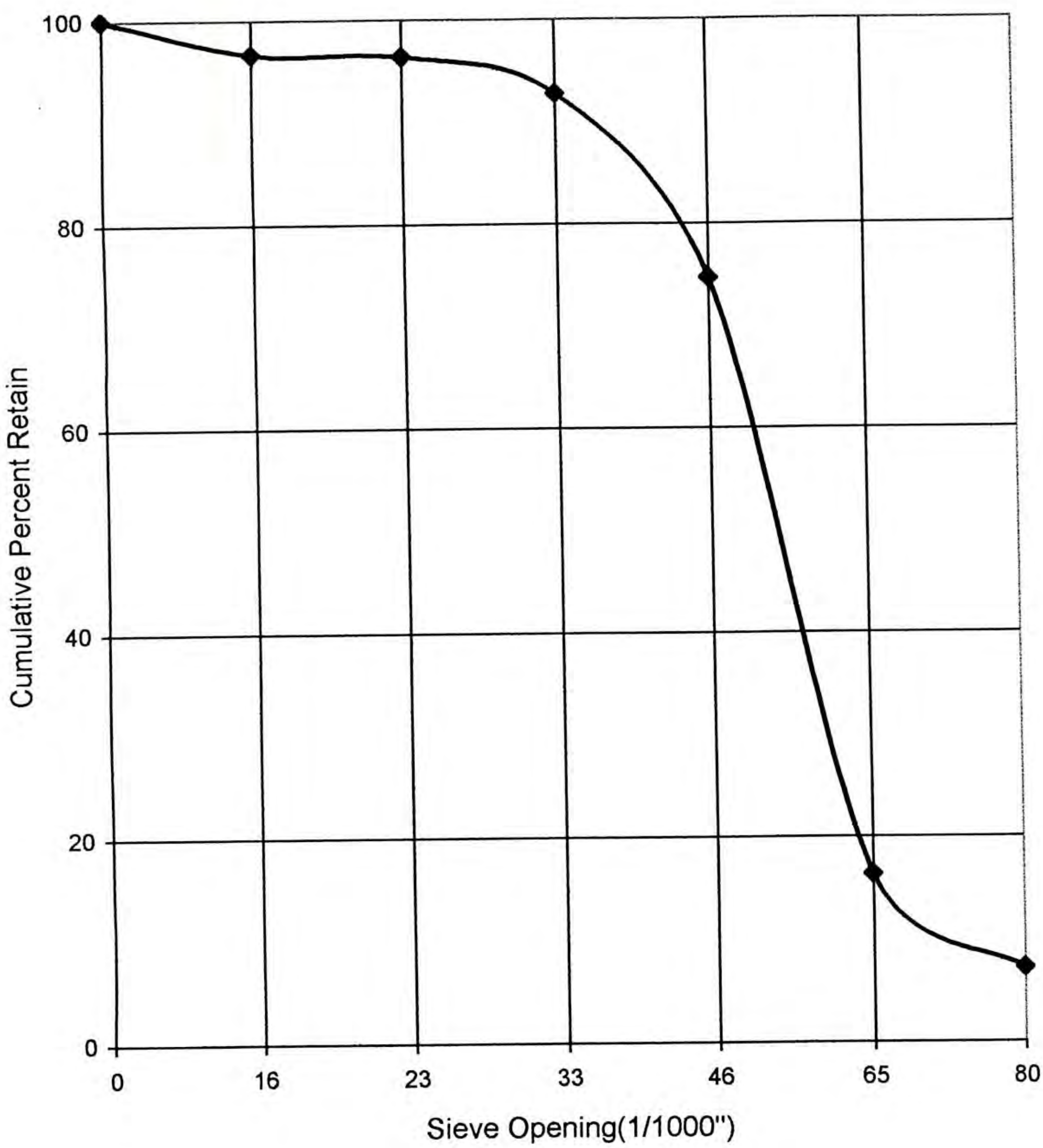
Sieve Analysis of Filter Materials of Hobigonj Treatment Plant



D_{10} = 40/1000" = 1.00 mm
D_{60} = 58/1000" = 1.45mm
U_c = 1.45

Figure E-2: Sieve Analysis of Filtering Materials, Hobigonj

Sieve Analysis of Filter Materials of Gopalgonj Treatment Plant



D_{10} = 36/1000" = 0.90 mm

D_{60} = 56/1000" = 1.40mm

U_c = 1.55

Figure E-3: Sieve Analysis of Filtering Materials, Gopalgonj

Sample : Sirajgonj

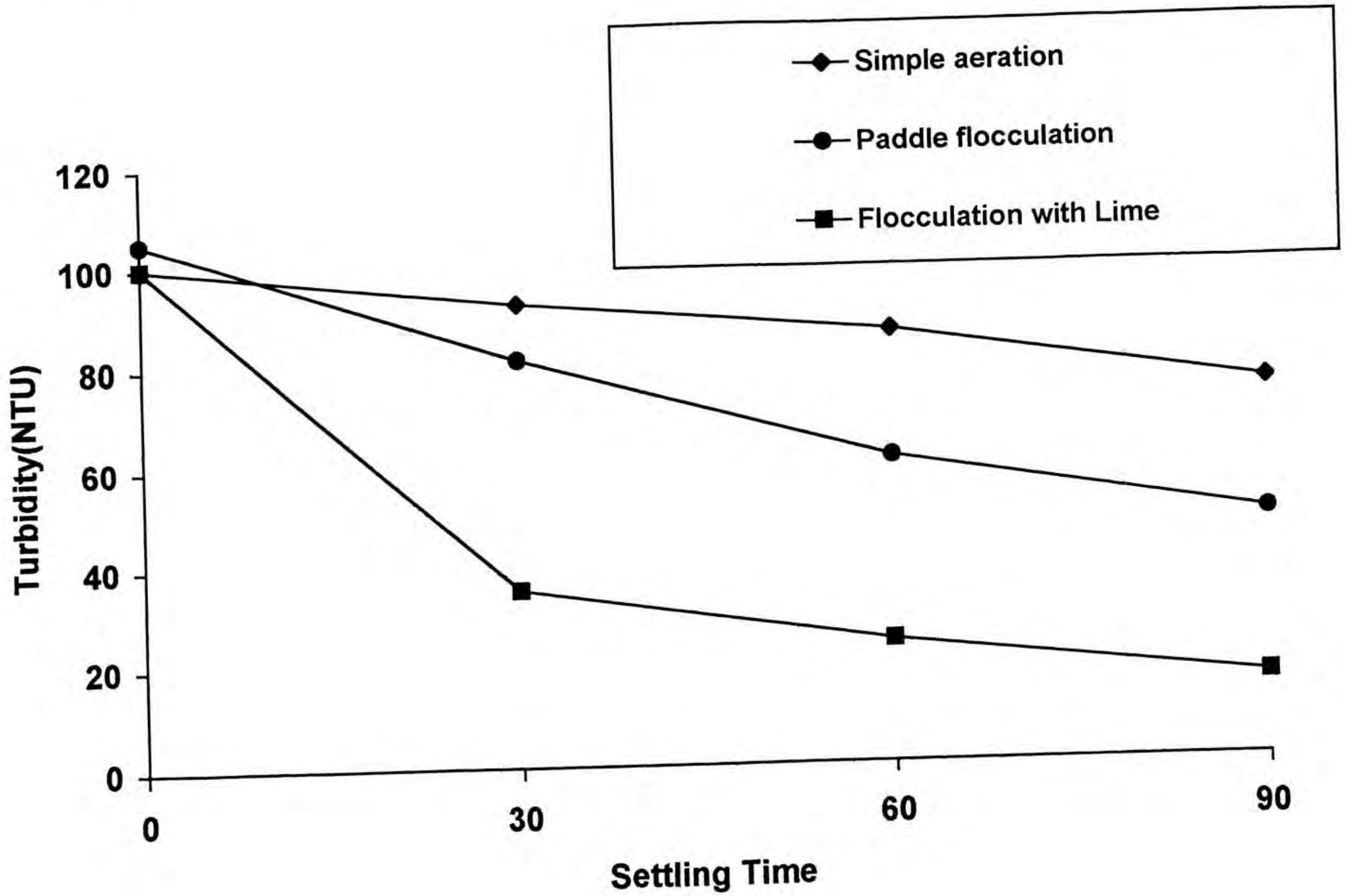


Figure F1: Settling Characteristics of Raw Water of IRP

Sample : Hobigonj

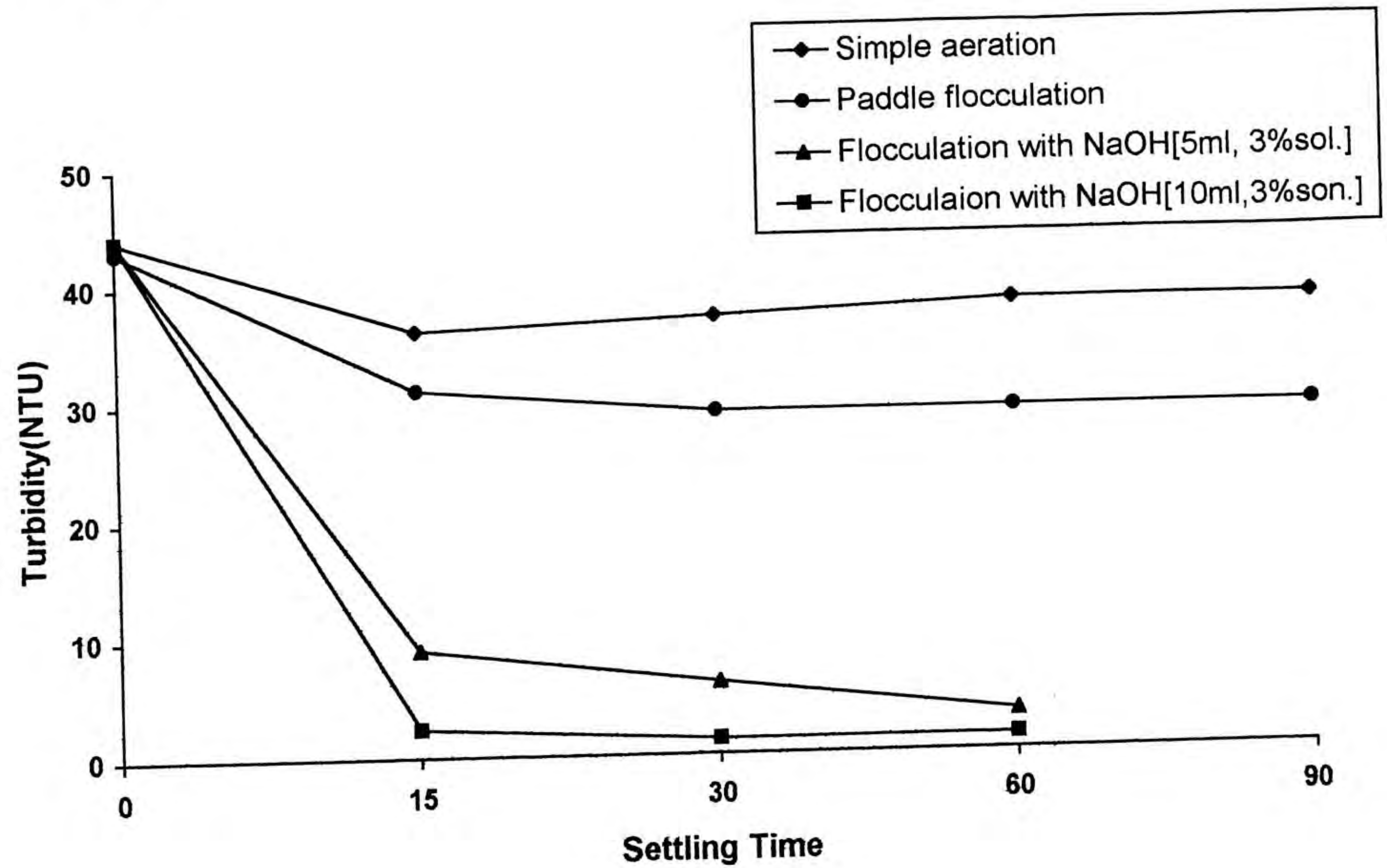


Figure F2: Settling Characteristics of Raw Water of IRP

Sample: Gopalgonj

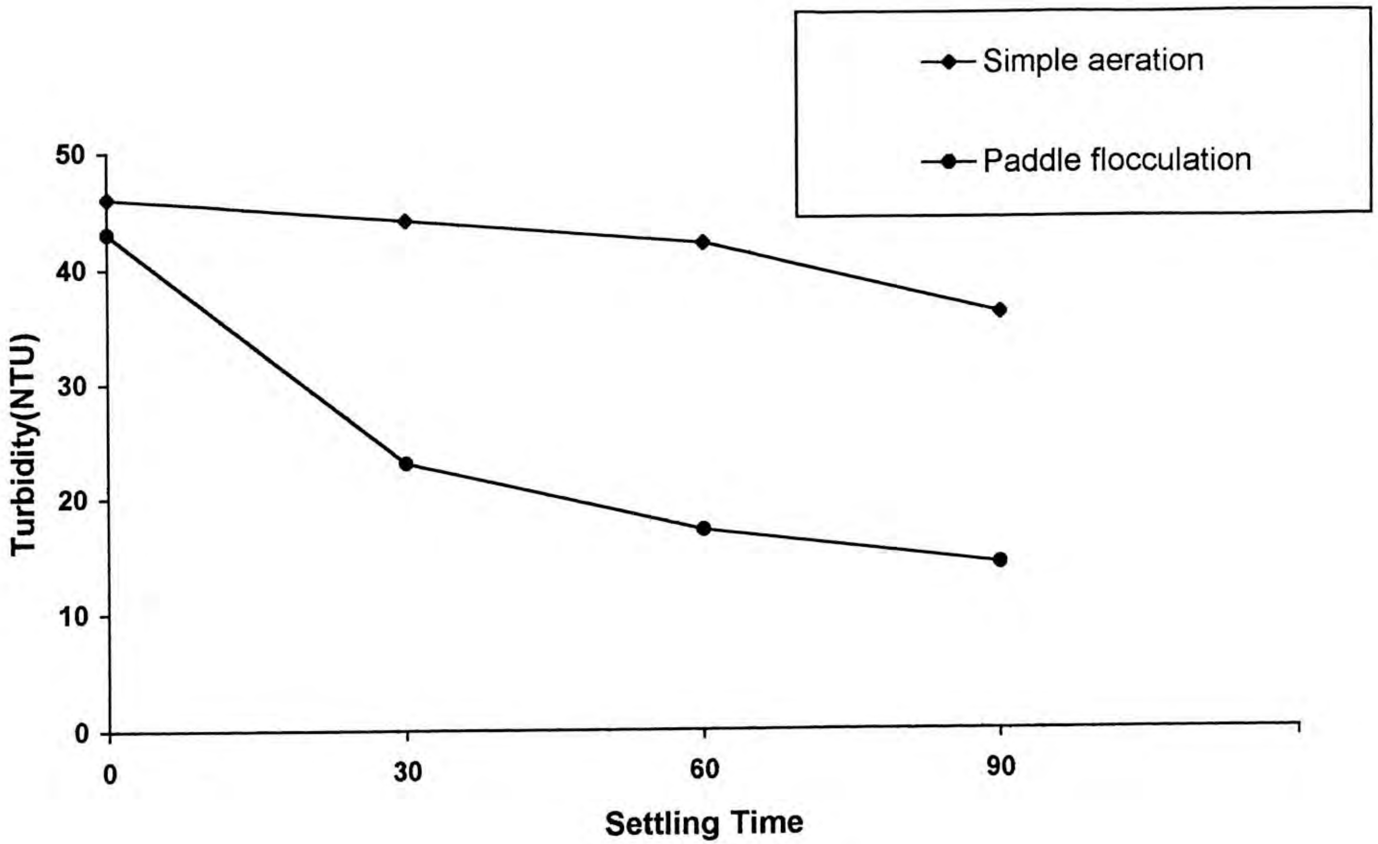


Figure F3: Settling Characteristic of Raw Water of IRP

Laboratory Model Test

Location : Hobigonj

	Raw	Aerated (compressed air)	Sample	Flocculation time(min)			Turbidity, NTU			
				50rpm	35rpm	20rpm	0 min	30 min	60 min	90 min
PH	7.1	7.75	1	5	3	5	38	28	28	28
CO_2	90	40	2	5	4	5	37	28	28	28
			3	5	3	6	40	29	29	29
				Unflocculated			44	36	37	38
			4	**50**	**35-40**	**15-20**	**5 min**	**15 min**	**30min**	**45min**
PH=10.5			5	5	3	7	22.5	9	6	3.2
PH=11			6	5	3	7	17.50	2.35	1.25	nm

- 5 ml NaOH was added to Sample 5. pH and alkalinity rose to 10.5 and 452ppm respectively.
- 5 ml NaOH was added to Sample 5. pH and alkalinity rose to 10.5 and 452ppm respectively.
- Flocculation: G= 90-100 sec-1 for 5 minutes; G=60-65 sec^{-1} for 3 minutes and G=25-35 sec-1 for 7 minutes total 15 minutes.
- Sedimentation: Optimum detention time ≈ 10 -12 minutes
- Turbidity load decrease to 22%

Location: Gopalgonj

PH	Raw	Aerated (compressed air)	Sample	Flocculation time(min)			Turbidity			
				50rpm	35rpm	15-20rpm	0 min	30 min	60 min	90 min
PH	7.3	7.6	1	5	5	2	45	30	23	17.6
CO_2	130	45	2	5	5	4	42	28	21	17.8
			3	5	5	6	43	23	17.1	14.2
			4	Non flocculated			54	44	42	36

- Flocculation: $G= 90\text{-}100\ sec^{-1}$ for 5 minutes; $G=60\text{-}65\ sec^{-1}$ for 5 minutes and $G=25\text{-}35$ sec-1 for 6 minutes; Total 16 minutes.
- Sedimentation: Optimum detention time ≈ 1.5 hours
- Turbidity load decrease to 33%

Location : Sirajgonj

	Raw	Aerated (compressed air)	Sample	Flocculation time(min)			Turbidity			
				50rpm	35rpm	20rpm	0 min	30 min.	60 min	90 min
pH	7.9-8.1	7.9-8.1	1	5	5	2	105	86	66	55
CO_2	50	20	2	5	5	4	81	81	68	60
			3	5	5	10	81	81	61	49
pH=9.6			4 (Add lime)	5	3	7	45	35	24	16

- 5 ml lime solution was added to Sample 4. pH rose to 9.6.
- Flocculation: G= 90-100 sec-1 for 5 minutes; G=60-65 sec^{-1} for 3 minutes and G=25-35 sec-1 for 7 minutes total 17 minutes.
- Sedimentation: Optimum detention time ≈ 1.5 hours.
- Turbidity load decrease to 35%

Appendix-H

O&M Cost Calculation of Proposed IRPs

In calculating the cost of O&M three major costs are considered, they are a)costs of Electricity, b)costs of Manpower and c)costs of maintenance. Since the IRPs are similar in type, capacity etc. It is assumed that production rates, cost on manmonth remains the same. In Sirajgonj and Hobigonj OHT tanks are proposed to use for backwashing. In Gopalgonj separate Backwash pump with a capacity of 550m3/hr. may be used. It is also assumed that the backwash requirement is 6%. The O&M costs are close to each other, however electrical cost varies as it depends on length of pump operation which in turn depends on supply hour, backwash time etc.

a) Electricity cost: The following emperical formula is used to calulate energy consumption of pumps. In this calculation the commercial rate of Tk.3.10 per Kwh unit of electricity is taken.

$$P(W) = \frac{2.78 * H * Q}{1000 * \eta}$$

Where, η efficiency (considered 65%)
Q discharge in m^3/sec
H head in m

Hobigonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk. 3.10 (Tk.)
Q=38.63 38.63 80.00	13 13 13	37.12 37.12 37.12	6.12 6.12 12.70	79.56 79.56 165.10	2386.80 22386.80 4953.00	7399.00 7399.00 15354.00
Q=160 (to OHT)	7.50(OHT) 1.25(BW)	26	17.80	155.75	4673	14485.00
						44,637.00

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.44,637*1.05*12=Tk.5,62,426.00 .

Sirajgonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk. 3.10 (Tk.)
Q=80 80	16 16	37.12 37.12	12.70 12.70	203.20 203.20	6096.00 6096.00	18897.60 18897.60
Q=160(to OHT)	7.50(Dist) 1.00(BW)	26	17.80	151.30	4539	14,071.00
						51,866.00

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.51,866*1.05*12=Tk.6,53,514/-.

Gopalgonj:

Pump rate M^3/hr.	Total pumping hr. (hr/day)	Head H(m)	P (kw)	P (kwh/day)	P (kwh/month)	Cost@Tk.3.10 (Tk.)
Q=50.22 (to aerator)	15.00	36.75	15.78	236.70	7101	22013
Q=148 (to OHT)	7.50	26	16.450	131.66	3949.82	12244.45
Q=550 (backwash pump)	0.20	10	23.52	4.70	141	437.50
						34,695.00

Assuming pump costs are 95% of the total electricity cost, the total yearly expenditure on electricity is Tk.34,695*1.05*12=Tk.4,37,157.00

b) Manpower:

The water supply set up of Sirajgonj is as follows:

Sl. No.	Name of the post	No. of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Pumpdriver	4	(1875-3605/-)	2.20
3.	Lineman	3	(1500/-2400/-)	0.90
4.	Bill clerk	1	(2550-5505/-)	0.70
5.	MLSS	1	(900-1530/-)	0.30
				5.10

Hobigonj

Sl. No.	Name of the post	No. of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Pumpdriver	4	(1875-3605/-)	2.00
3.	Lineman	3	(1500/-2400/-)	1.27
4.	Bill clerk	1	(2550-5505/-)	0.70
5.	MLSS	1	(900-1530/-)	0.30
				5.39

Gopalgonj:

Sl. No.	Name of the post	No of Post	Pay scale Tk.	Annual cost (in lakh Taka)
1.	Superintendent	1	(3400-6625/-)	1.00
2.	Account Assistant	1	(2550-5505/-)	0.70
3.	Pumpdriver	5	(1875-3605/-)	2.50
4.	Electrician	1	(1500-2400/-)	0.45
5.	Lineman	1	(1500/-2400/-)	0.45
6.	Bill clerk	1	(2550-5505/-)	0.70
7.	MLSS	1	(900-1530/-)	0.30
				6.10

c) Depreciation cost of pumps

considering depreciation rate 10%/year (10year design life)
6,00,000x0.10 =**Tk.60,000**
For Gopalgonj one extra backwash pump with capacity of 50m^3/h cost about 5.00lakh, so for this town depriciation cost is 1.10lakh

d)Other

i) Maintenance

The yearly maintenance cost are assumed to be : 3% of investment cost for pipe line, mechanical and electrical parts and 1% of investment cost for civil works. It is also assumed that the cost for civil and E&M cost are 65% and 35% of total investment cost of the system respectively.

The investment cost of the system was 100.00 lakh Taka (including 18lakh for flocculator and settling tank.

Share for the civil works is 100*0.65=65 lakh taka and that for E&M works is 35lakh taka.

So, Yearly Maintenance cost will be

1) for civil works 65*0.01=0.65 lakh taka.
2) for E&M works 35*0.03=1.05 lakh taka.

Total = 1.70 lakh taka

ii)Cost of replacing sand (biannual) :

amount of sand for 2 filter = 2*13.5*1.5 = 40.80cum

so yearly expenditure for replacement of sand @ Tk.2328/-/2 = **Tk.47500/-**

Total cost others: 1.70+0.47 = Tk. 2.10lakh

Cost of Production:

i)Sirajgonj:

Total yearly cost of Operation and Maintenance: (a+b+c+d)=6.58+5.1+0.60+2.10
=Tk.14.38 lakh

Total water production: 2720 m^3/day[*ref :appendix-A*]
Total production: 2720m^3/day*30day*12 = 9,79,200m^3/year
Total backwash amount @ 6% of total production: 58752m^3/year.
Net monthly production: 979200-58752=9,20,448 m^3
So production cost comes to : Tk.14.38lakh/9.20lakhm^3=Tk.1.56/m3

ii)Hobigonj:

Total yearly cost of Operation and Maintenance: (a+b+c+d)=5.66+5.33+0.60+2.10
=Tk.13.69 lakh

Total water production: 2619.95m^3/day [*ref :appendix-A*]
Total production: 2619.95m^3/day*30day*12=942840m^3/yr.
Total backwash amount @ 6% : 56570m^3

Net monthly production: 942840-56570=886269m^3

So production cost comes to : 13.69 lakh Taka/8.86lakhm^3=Tk.1.54/m3

iii)Gopalgonj:

Total yearly cost of Operation and Maintenance: (a+b+c+d) = 4.43+6.10+1.10+2.10
=Tk.13.73 lakh

Total water production: 1506m^3/day [*ref :appendix-A*]

Total production: = 5,42,160m^3/yr.

Total backwash amount @ 6% : 32529m^3

Net production: 542160-32529=5,09,631m^3

So production cost comes to : 13.73 lakh Taka/5.096lakh m^3 = Tk. 2.60