Tertiary Wastewater Treatment by Sedimentation & Sand Filtration

by

Suhail Ahmad

A Thesis Presented to the

FACULTY OF THE COLLEGE OF GRADUATE STUDIES

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DHAHRAN, SAUDI ARABIA

In Partial Fulfillment of the
Requirements for the Degree of

MASTER OF SCIENCE

In

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Tertiary wastewater treatment by sedimentation and sand filtration

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This thesis, written by Mr. Suhail Ahmad under the direction of his Thesis Committee, and approved by all its members, has been presented to and accepted by the Dean, College of Graduate Studies, in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

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بسم الله الرحمن الرحيم

THIS THESIS IS DEDICATED TO

my mother

BEGUM MASROOR JAHAN

and

my brother

Mr. AFTAB AHMAD

ACKNOWLEDGEMENT

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ABSTRACT

Tertiary treatment was given to the secondary wastewater effluent of the North ARAMCO Wastewater Treatment Plant Dhahran in order to utilize it for unrestricted reuse and landscape irrigation. The process employed for this study consisted coagulation, flocculation, sedimentation, granular media filtration and chlorination. A bench-scale study was conducted to optimize coagulant dosages and filtration rate to produce a satisfactory filtered effluent for chlorination. Based upon the experimental data, a coagulant dosage and filtration rate were suggested for the tertiary treatment of two kinds of wastewater effluent (chlorinated and unchlorinated). A mathematical relationship was developed based upon the batch chlorination data to determine the chlorine residual and contact time to achieve an effluent coliform concentration of less than 2.2 MPN/100 ml that is safe for unrestricted reuse as described by Title 22 of California Administrative Code.

موجـــــــــــز

تمت المعالجة الثالثة لمياه مجارى محطة شمال الظهران فى ارامكـــــــو والمعالجة (بيولوجيا) درجة ثانية لاعادة استعمالها بشكل غير مقيد لــــــرى الحدائـــــق ٠

ان العمليات التى وضعت فى هذه المعالجة تتالف من : التخثير، التجميـــع الترسيب ، الترشيح فى وسط حبيبى وثم التعقيم بالكلـــــوز ٠

تمت الدراسة على جهاز مصفى لتصميم الجرعات الامثلية للمخثرات ومعـــــــدل الترشيح للحصول على مياه مرشحة جيدة للتعقيـــم ٠

وحسب النتائج التى تم الحصول عليها فقد اقترحت الجرعة الامثليـــــــــــــة للمخثرات مع معدل للترشيح لنوعية من مياه مجارى ناتجة عن معالجة بيولوجيـــــة (معقمة وغير معقمـــــة) ٠

وتم ايجاد علاقة رياضية اعتمادت على نطاق مغلق للتعقيم بالكلوز لتقديـــم كمية الكلوز المتبقى من التعقيم للحصول على تركيز الكوليفورم فى الميـــــــاه الناتجة اقل من ٢ر٢ كوليفورم /١٠٠ ٠ وهو الذى يعتبر آمن لاعادة استعمــــــال المياه الناتجة بشكل غير مقيد كما ورد فى ٢٢ من مصطلح كاليفورنيا الادارى ٠

Chapter 1

INTRODUCTION

1.1 BACKGROUND OF TERTIARY TREATMENT

Tertiary wastewater treatment is additional processing to remove pollutants that are not adequately removed by conventional biological techniques. These pollutants may be soluble inorganic compounds like nitrogen and phosphorus, which are responsible for eutrophication of lakes or streams; organic materials contributing biochemical oxygen demand (BOD), chemical oxygen demand (COD), color, taste and odor; bacteria; viruses; colloidal solids shielding harmful microorganisms and contributing turbidity; or soluble minerals that may interfere with subsequent reuse of the wastewater. The aim of tertiary treatment is usually, not to remove all of these pollutants completely but to reduce their concentrations to acceptable limits as per standard imposed by the regulatory authorities. Water demand is increasing due to growing population that has already created pollution problems at many locations that cannot be solved by secondary treatment alone. The number of these instances are increasing and reuse of treated wastewater has become necessary in order to meet the water demand and to keep up the economy.

Characteristics of conventional wastewater effluent are defined in terms of BOD, suspended solids and fecal coliform concentrations. In a period of 30 consecutive days measurement, the arithmetic mean is normally limited to 30 mg/l for BOD and suspended solids each while the geometric mean concentration of fecal coliform is to be less than 200 per 100 ml. These standards are not adequate for direct human contact and tertiary treatment is required to further reduce BOD, concentration of suspended solids, and viral, bacterial, protozoan and helminth pathogens.

The general process scheme for tertiary treatment consists of coagulation, flocculation, sedimentation, filtration and disinfection. Several other combinations can be employed depending on the types of pollutant to be removed. The size of particulate impurities in water vary from a few Angstroms to a few hundred microns. In water and wastewater treatment, most of the impurities are removed by sedimentation. However some are too small for gravitational settling alone and need aggregation of small particles into large floc for successful separation by sedimentation. This process of aggregation is known as coagulation. The most important factor for coagulation is a proper dosage of coagulants, with or without a coagulation aid. Choice of coagulant is also an important consideration. Aluminium sulfate (alum) is one of the most effective and economical coagulants and is extensively used in the water treatment. Alum coagulation and flocculation is affected by many factors such as pH, temperature, nature of colloids, size of turbidity particles, mixing and alum concentration. The latter two factors can be easily adjusted to an optimal condition in the design stage. Coagulant aids improve alum flocculation, if a proper amount is used together with a suitable application sequence. A jar-test study is essential to confirm the effectiveness of chemical dosages.

Particulate matter in secondary effluent contains a significant fraction of the microorganisms contained in the untreated wastewater effluent. Viruses and bacteria entrained in the particles may be shielded from a disinfectant. Therefore, some form of clarification prior to disinfection is necessary to reliably achieve high levels of microorganisms inactivation. Filtration is utilized to enhance the efficiency and reliability of subsequent disinfection by lowering the levels of suspended solids and turbidity. Also coagulation and filtration remove worm ova and protozoan cysts that are resistant to chlorination. A water having a turbidity less than 1 nephelometric turbidity units (NTU) (1) and a coliform concentration of 2.2 per 100 ml or less is considered safe for unrestricted reuse (2).

1.2 STATEMENT OF THE PROBLEM

Potential water shortage is an acute problem in Saudi Arabia. Water demand is increasing day by day due to rapid population growth, industrialization and agricultural activities. Particular stress is being given to cultivate green public parks for recreation and beautification of cities. Making green areas is a very difficult task in an arid region like Saudi Arabia, requiring an abundant quantity of water for maintenance. So, water reuse is essential for meeting the water demand.

The North ARAMCO Wastewater Treatment Plant is designed to treat 8.0 million gallons of wastewater per day. This plant utilizes the extended aeration process mode to produce a high-quality secondary effluent. The treated wastewater after the chlorination is pumped via a 9-mile pipeline to a site south of Dhahran where the effluent is percolated into the soil in a series of percolation ponds. Because of the limited permeability of the soil, this operation requires continuing management.

Moreover, this 8.0 million gallons of water per day is being disposed off without regard to its value. Reuse of this treated wastewater is appropriate for landscape irrigation after tertiary treatment to an acceptable water quality as specified in Title 22 of California Administrative Code (2).

1.3 OBJECTIVES

The specific objectives of this study were to :

(a) Determine the treatability of the two kinds of wastewater effluents from the ARAMCO Wastewater Treatment Plant by tertiary treatment using sedimentation and sand filtration processes.

(b) Conduct a bench-scale study to optimize coagulant dosages and filtration rate to produce a satisfactory filtered effluent for disinfection.

(c) Perform batch disinfection tests on the filtered effluent to determine contact time and chlorine residual concentration necessary to reduce the coliform concentration to 2.2 per 100 ml.

Chapter 2

LITERATURE REVIEW

Secondary effluent from conventional treatment is not safe for human contact even if it is chlorinated since viruses have been detected in chlorinated secondary effluents. Title 22 of California Administrative Code (2) states : "Reclaimed water used for the irrigation of parks, playgrounds, schoolyards and other areas where the public has similar access or exposure shall be at all times an adequately disinfected, oxidized, coagulated, clarified, filtered wastewater or a wastewater treated by a sequence of unit processes that will assure an equivalent degree of treatment and reliability. The wastewater shall be considered adequately disinfected if the median number of coliform organisms in the effluent does not exceed 2.2 per 100 millilitres, as determined from bacteriological results of the last 7 days for which analysis have been completed, and the number of coliform organisms does not exceed 23 per 100 millilitres in any sample". Because viruses normally occur in very low concentrations and their assay needs special expertise, virus monitoring is not required in this criterion. Filtration preceded by chemical coagulation and sedimentation can remove protozoal cysts, worm ova and suspended solids that protect bacteria and viruses from the oxidizing action of chlorine. Filtered secondary effluent after chlorination can be used for unrestricted irrigation of landscape and food crops, ground water recharge by surface spreading and for recreational purposes.

2.1 REUSE OF TREATED WASTEWATER FOR IRRIGATION AND GROUNDWATER RECHARGE

The application of reclaimed water for irrigation has a long history, originating in England in the nineteenth century where sewage farms and sewage collection systems were first developed. In the United States, the practice of using raw sewage for irrigation was abandoned in the early part of this century, and by the 1930's a minimum of primary treatment was required before water could be used for agriculture (3). A survey of Municipal Wastewater Reuse by Schmidt (4) indicated that 358 locations, mostly in the semi-arid southwest U.S., reuse wastewater for such purposes as irrigation, industrial cooling and process water, recreational lakes, and fish propagation. About 60 % of the total wastewater is reused for agricultural irrigation, 30 % for industrial cooling and process waters, and the remainder 10 % of the total wastewater is used for fish and wildlife, recreation and groundwater recharge. The advantages in the use of treated wastewater for irrigation are (i) low-cost, (ii) an economical way to dispose of wastewater, (iii) an effective use of nutrients present in wastewater, and (iv) providing additional treatment before recharging the ground water reservoir.

Kaufman (5) reviewed pollution of groundwater through the application of wastes to soil and concluded that such pollution is not a widespread problem. Organics, nitrogen, minerals, detergents, pesticides, petroleum and heavy metals were covered. Fetter and Holtzmachers (6) also reviewed the general topics of groundwater recharge and concluded that, with secondary effluent, recharge by spray irrigation or spreading basins is possible but recharge wells require much higher quality water. Injection of high-quality tertiary effluent into the groundwater was reported by Faust and Vecchioli (7). Although secondary effluent was treated by alum coagulation, sedimentation, filtration, carbon adsorption, and chlorination, clogging of injection well occurred. Over a 30-day period, specific capacity declined 50 percent. Precipitation of ferric phosphate and / or growth of bacteria around the well screen were suggested as possible causes. Schmidt, et al. (8) reviewed the variety of formal and informal additions of wastewaters to groundwater. Examples of groundwater recharge by direct injection in Orange County, City of Los Angeles, and Santa Clara Valley, California, Nassau County, N.Y. and the City of Chicago, Illinois were reported. Idelovitch (9) demonstrated the use of oxidation ponds followed by lime-magnesium treatment to produce an effluent of high enough quality for groundwater recharge. The high pH treatment followed by polishing ponds resulted in ammonia stripping, and removal of phosphorus, organics, trace elements, bacteria and viruses. Quality was further improved by the variety of processes taking place in the soil and in the aquifer. Reclaimed water quality was used for unrestricted crop irrigation and industrial uses.

2.2 POTENTIAL HEALTH EFFECTS ASSOCIATED WITH THE REUSE OF WASTEWATER

When sewage effluent is to be used for irrigation, the primary public health consideration is the prevention of diseases caused by viruses, bacteria, protozoa, and helminths that could be present in the effluent in great numbers and variety. Table 2.1 gives some examples of water related diseases (10). Several states have formulated quality criteria for irrigation with sewage effluent. These criteria are developed on the basis of what could be in the effluent and what might happen to the people than on documented evidence of disease caused by irrigation with effluents (3). Thus, although a good secondary disinfected effluent with fecal coliform concentration less than 1000 per 100 ml may be suitable for restricted irrigation, state criteria normally are much more stringent. For unrestricted irrigation, for example, the State of California (2) requires that the effluent be well oxidized, coagulated, clarified, filtered and adequately disinfected having 7-day median coliform concentration not in excess of 2.2 per 100 ml and the maximum coliform concentration during a 30-day period shall not exceed 23 per 100 ml. The same standards apply to irrigation of parks and playgrounds. For irrigation of fodder, fibre, and seed crops and or orchards and vineyards, primary treatment is sufficient. Irrigation of pasture for milking animals requires an oxidized and disinfected effluent with a coliform count of less than 23 per 100 ml. This effluent is also suitable for landscape irrigation (2).

The relative reduction of coliform organisms generally serves as an indication of the microbiological efficiency of wastewater treatment processes. In primary sedimentation, a reduction of 30-40 % in the number of coliform organisms is obtained, while in most full biological

Table 2.1 : Some Examples of Water-Related Diseases

Catagory	Type of pathogenic agents	Disease
1. Water borne	Bacteria	Bacillary dysentry Cholera Leptospirosis Typhoid and Paratyphoid fevers
	Virus	Infectious hepatitis Enterovirus infections
	Protozoa	Amoebic dysentry Giardiasis
	Nematodes (roundworms)	Ascariasis
2. Water-washed	Bacteria	Leprosy Yaws
	Fungus	Tinea(ringworm)
	Rickettsiae	Typhus
3. Water-based	Cestode(tapeworm)	Diphyllobothriasis
	Trematodes(flukes)	Fascioliasis Schistosomiasis
4. Water-related	Virus	Arborviral infections Denque Yellow fever

treatment processes the reduction is between 90 to 95 %. Most vegetative bacterial pathogens appear to be removed in the same proportion as coliform organisms. Certain helminth eggs having rapid sedimentation rates may be effectively removed by conventional primary sedimentation treatment and even more effectively removed by stabilization pond treatment of 5-7 days retention. Viruses are less effectively removed by conventional wastewater treatment processes and may remain in a chlorinated effluent even when coliforms have been reduced appreciably. Ozone disinfection is particularly effective against viruses but is rarely practised (11).

2.3 TERTIARY WASTEWATER TREATMENT PERFORMANCE

Removal of viruses by various tertiary treatment systems was the subject of the Pomona Virus Study (12). Flow diagrams for three of the systems are given in Fig. 2.1. The first system was the system required by Title 22 of California Administrative Code (2) for treating wastewater for unrestricted recreational reuse that included coagulation with an alum dosage of 150 mg/l and polymer dosage 0.2 mg/l followed by sedimentation, dual media filtration at 3.4 $l/m^2.s$ (5 gpm/ft^2), and chlorine contact for 2 hr with minimum residuals of 5 and 10 mg/l. The second system consisted direct filtration (without flocculation or sedimentation) with an alum dosage of 5 mg/l and polymer dosage 0.06 mg/l followed by filtration and chlorination same as first system. The third system consisted carbon adsorption with intermediate chlorination. Contact time with both granular carbon bed was 10 min while the first

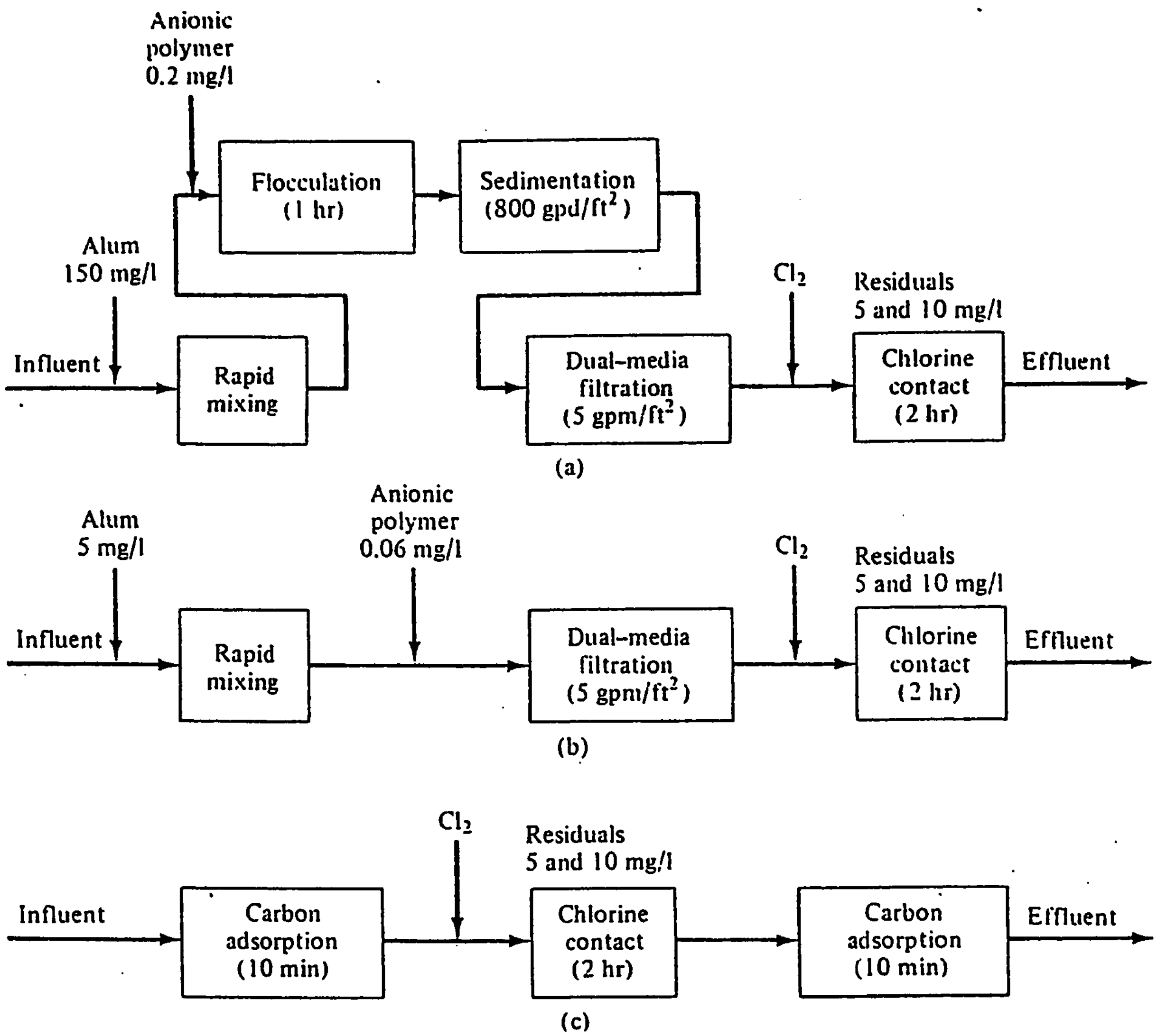

Fig. 2.1 : Three of the Tertiary Treatment Systems Investigated for Virus Removal Efficiency in the Pomona, CA. Pilot-Plant Studies (a) Coagulation, Sedimentation, Filtration, and Disinfection (b) Coagulation, Filtration, and Disinfection (c) Two-stage Carbon Adsorption and Disinfection (12)

bed provided a gravity filtration of 2.4 $l/m^2.s$ (3.5 gpm/ft^2) and the other provided 2.7 $l/m^2.s$ (4.0 gpm/ft^2). The major objectives of the study were (a) to rank the treatment systems in terms of virus removal efficiency and reliability and (b) to derive cost estimates for systems performing equivalently to the required system with respect to viral removal (12).

Naturally occurring virus concentrations were very low and rare, therefore, seeding experiments were conducted with sufficient virus concentrations to produce measurable effluent virus counts. The conclusions derived from the study were that the majority of virus inactivation occurred during disinfection and the main function of the unit processes preceding disinfection was to remove substances that interfere with efficient disinfection. The effluent virus concentration was directly affected by the magnitude of chlorine residual. The least cost system that performed equivalently to the required system was direct filtration with disinfection at the higher chlorine residual of 10 mg/l.

A four-year study was performed in which enteroviruses were monitored at a full scale 56 ML/d advanced wastewater treatment plant, Orange County (California) Water District's Water Factory 21 (13). The process included lime treatment, air stripping, recarbonation, prechlorination, mixed media filtration, granular activated carbon adsorption, final chlorination, and reverse osmosis demineralization. The viral burden entering Water Factory 21 when the influent was effluent from a trickling filter was compared with that for activated sludge effluent. In both cases, the distribution was log normal with mean values of 1.3

PFU/L (plaque-forming units per litre) for the trickling filter effluent and 0.13 MPNCU/L (most probable number of cytopathic unit per litre) for the activated sludge. The activated sludge process was more efficient than the trickling filters in removing viruses. Removal of viruses by lime precipitation was observed to be approximately 2 logs (98.8 percent). Viral analysis in the effluent of granular activated carbon and reverse osmosis processes was negative, whereas only 2 out of 215 chlorinated samples were positive in viral analysis.

2.4 COAGULATION AND FLOCCULATION

The terms "coagulation" and "flocculation" have some different interpretations in the chemical and engineering literature. According to Lamer (1964), coagulation refers to destabilization produced by the compression of the electric double layers surrounding all colloidal particles, while flocculation refers to destabilization by the adsorption of large organic polymers and the subsequent formation of particle-polymer-particle bridges (14). Two theories have been advanced to explain the basic mechanisms involved in the stability and instability of colloid systems : (1) chemical theory assumes that colloids are aggregates of definite chemical structural units, and proposes that coagulation occurs because of specific chemical reactions between the colloidal particles and the chemical coagulant added. (2) Physical theory proposes that reduction of forces tending to keep colloids apart occurs through the reduction of electrostatic forces, such as zeta potential. The two theories are not mutually exclusive and both must be employed to explain the operation of an effective chemical coagulation process (15).

Stumm and O'Melia (16) reviewed some chemical factors effective in destabilization of colloids and concluded that coagulation is a time dependent process including several reaction steps : (a) hydrolysis of multivalent metal ions and subsequent polymerization to multinuclear hydrolysis species; (b) adsorption of hydrolysis species at the solid-solution interface to accomplish destabilization of the colloids; (c) aggregation of destabilized particles by interparticle bridging involving particle transport and chemical interactions; (d) aggregation of destabilized particles by particle transport and Van der Waal's forces; (e) "aging" of flocs, accompanied by chemical changes in the structure of metal-OH-metal linkages, concurrent change in floc sorbability and in extent of floc hydration; and (f) precipitation of metal hydroxide.

2.4.1 Optimization of Coagulation for Sedimentation

The use of alum as a coagulant in the purification of water is well established and many studies have been conducted to optimize and understand its use. Black and Hannah (17) pointed out that, the coagulation of turbid water may be affected by many variables including (i) the type, amount and size distribution of turbidity, (ii) specific ions present, (iii) pH, (iv) coagulant type and dosage and (v) alkalinity. Jeffcoat and Singley (18) studied the effect of alum concentration and chemical addition times on coagulation and concluded that the optimum pH for coagulation increased when the alum dosage increased. Also better turbidity removal occurred with dilute alum solutions.

Bratby (19) reported optimum pH value as 7.6 at alum dosage greater than 5 mg/l and variation in optimum pH from 6.4 at 2 mg/l to 7.6 at 5 mg/l. If primary coagulants were used alone, optimization was best based on the total cost of coagulant and pH adjustment chemicals and associated cost of sedimentation. When primary coagulant was applied with flocculant aids, optimization of the primary coagulant was best based on the consideration of downstream processes like filtration, with optimization of the coagulant aid based on settling considerations. In another study (20), he interpreted laboratory results for the design of rapid mixing and flocculation systems and found that a full scale design of rapid mixing and flocculation systems could be developed from batch laboratory results.

2.5 SEDIMENTATION

Sedimentation is a solid-liquid separation utilizing gravitational settling to remove suspended solids. In advanced wastewater treatment and tertiary treatment, the main purpose of the sedimentation is to remove chemically coagulated floc prior to filtration. It is one of the earliest unit operations used in water and wastewater treatment. The settling characteristics of the floc depend upon the characteristics of the water and wastewater, the coagulant used, and the degree of flocculation. The only method to accurately determine the settling velocities and the required overflow rates and detention time is to perform experimental settling tests. Generally overflow rates of 20.4 to 24.4 $m^3/m^2.d$ (500 to 600 gal/day-ft^2) and the detention times of 2 to 8 hr are used for wastewaters coagulated with alum (21).

Particles settle from suspension in different ways, depending upon the concentration of the suspension and the characteristics of the particles. The four distinct types of sedimentation which reflect the concentration of the suspension and the flocculating properties of the particles are as follows : "Class-1 Clarification" is the settling of a dilute suspension of particles which have little or no tendency to flocculate. The removal of a dilute suspension of flocculant particles is referred to as "Class-2 Clarification". When particles are sufficiently close, interparticle forces are able to hold them in fixed positions relative to each other. As a result, the particles subside as a large mass rather than as discrete particles. This type of clarification is called "zone settling". Figure 2.2 illustrates zone settling for intermediate concentrations of flocculant particles and higher concentrations of more particulate suspensions. When the particles actually contact each other the resulting structure of the compacting mass acts to restrict further consolidation, this action is called "compression" (14).

2.6 GRANULAR MEDIA FILTRATION

The most common method of removing colloidal impurities in water processing and tertiary wastewater treatment is gravity filtration through beds of granular media. Granular media used for water and wastewater filters include sand, crushed anthracite coal, diatomaceous earth, perlite and granular activated carbon. Combinations of these media are also in use. However, the most common medium is a graded

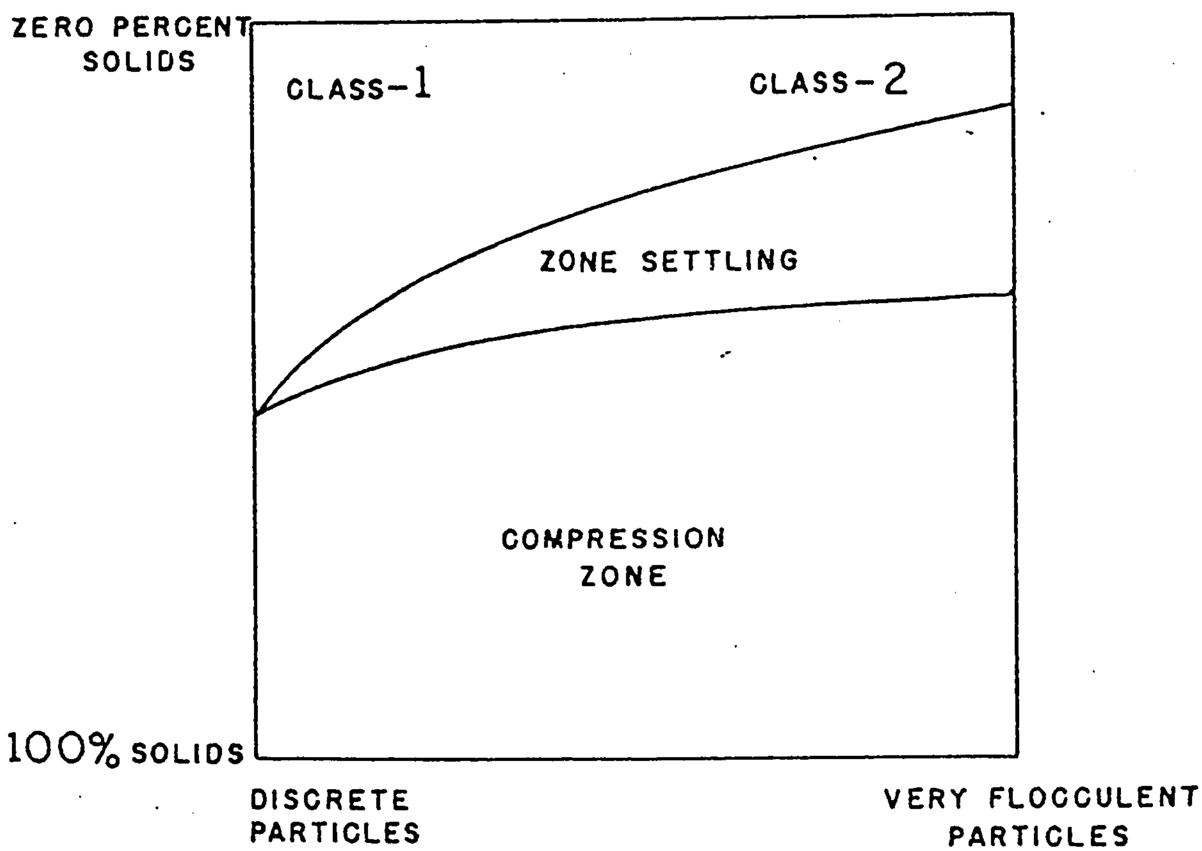

Fig. 2.2 : Types of Sedimentation (14)

bed of silica sand.

The development of sand filter took place in England in the mid nineteenth century. These filters were operated at the relatively low rates of 0.03 to 0.08 $l/m^2.s$ (0.04 to 0.12 gpm/ft^2), and generally functioned satisfactorily on untreated English surface waters. These filters were not generally successful on American waters, and this led to the development of preceding sand filtration by coagulation. United States filters, developed in the late nineteenth century, were operated at much higher rates, ranging from 0.68 to 2.72 $l/m^2.s$ (1.0 to 4.0 gpm/ft^2). The higher rates used in United States filter meant less filter area and less capital investment to achieve desired capacity. These filters are commonly called rapid sand filters to contrast them to the English or slow sand filters (14).

2.6.1 Principal Mechanisms of Filtration

The mechanisms involved in removing suspended solids in a granular media filter are very complex, consisting of interception, straining, flocculation, and sedimentation as shown schematically in Fig. 2.3 (22). Many studies discuss the various factors that may play an important role in removal. The dominant mechanisms depend on the physical and chemical characteristics of the suspension and of the medium, the rate of filtration, and the chemical characteristics of the water.

With granular bed filters, removal is primarily within the filter bed, commonly referred to as depth filtration. Efficiency of depth removal depends on a number of mechanisms. Some solids may be

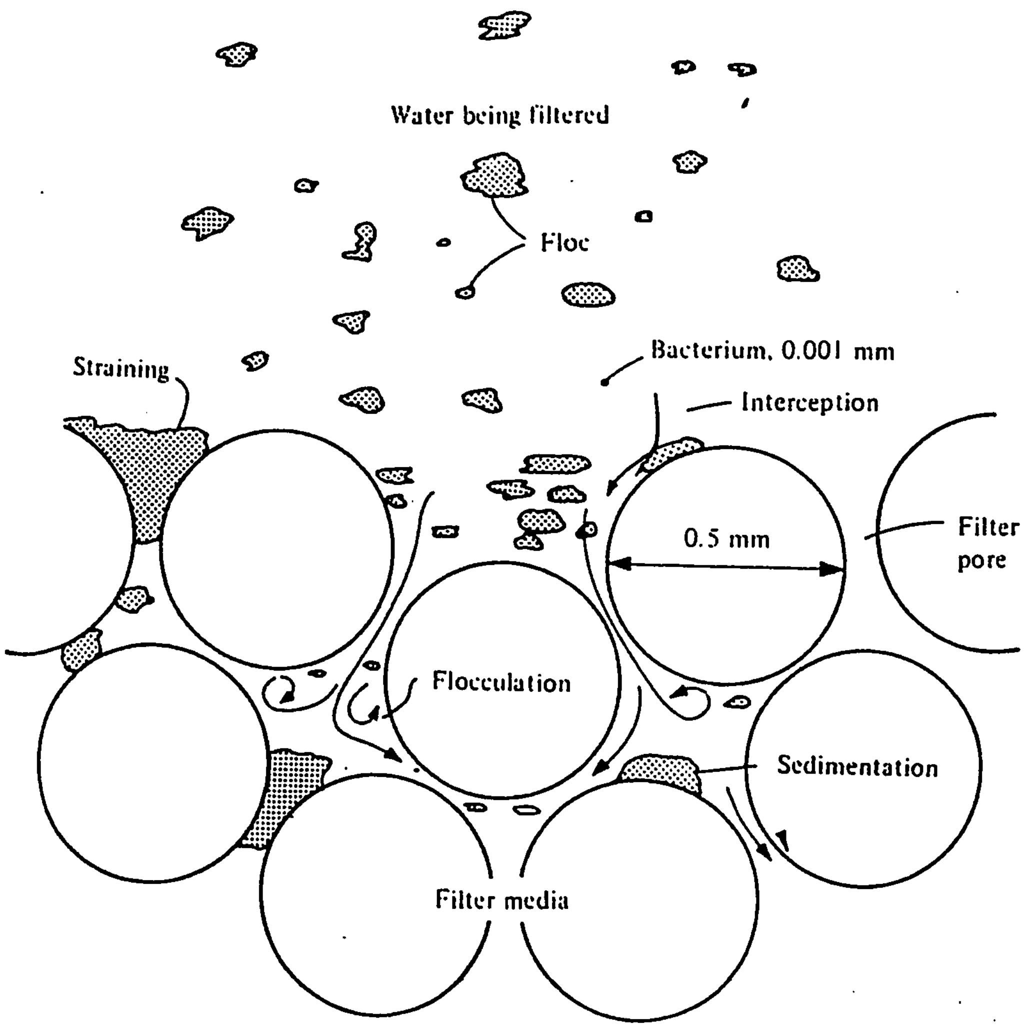

Fig. 2.3 : Schematic Diagram Illustrating Straining, Flocculation, and Sedimentation Action in a Granular-Media Filter (22)

removed by the simple mechanical process of interstitial straining. Removal of other solids, particularly the smaller solids, depends on two types of mechanisms. First a transport mechanism must bring the small particle from the bulk of the fluid within the intertices close to the surfaces of the media. Transport mechanisms may include gravitational settling, diffusion, interception, and hydrodynamics; these are affected by such physical characteristics as size of the filter medium; filtration rate; fluid temperature; and the density, size, and shape of the suspended particles (23).

Second, as the particle approaches the surface of the medium, or of previously deposited solids on the medium, an attachment mechanism is required to retain the particle. The attachment mechanism may involve electrostatic interactions, chemical bridging, or specific adsorption, all of which are affected by the coagulants applied in the pretreatment as well as by the chemical characteristics of the water and of the filter medium (23).

During filtration, surface straining and interstitial removal clogs the upper portion of the filter media. The velocity of the water through the remaining voids increases, due to the reduction in pore area, shearing off pieces of captured floc and carrying impurities deeper into the filter bed. The burden of removal passes deeper and deeper into the filter. Ultimately, clean bed depth is inadequate to provide the desired effluent quality and causing termination of the filter run.

2.6.2 Filter Performance for Optimum Design

In common practice, the two criteria for terminating the filtration run are :

i) The effluent quality criterion, expressed in terms of the maximum permissible filter effluent turbidity, or suspended solids concentration.

ii) The headloss criterion, that is the maximum headloss allowed to develop across the filter.

The two criteria are arbitrary and are determined for each treatment plant to meet local specific sanitary, economic and hydraulic needs. Adin (24) used the principle of the optimum model to compare the use of alum and cationic polyelectrolytes for chemical pretreatment on the basis of optimum run output and corresponding filter bed depth values of head loss. He concluded that the design of granular deep bed filters can be aided accepting the principle that the filtration process, similarly to other accumulation processes, is characterized by an advanced clogging front and by breakthrough curves. Application of the principle can play a key role in filter design. Consequently, an accumulation-detachment model which incorporates filter capacity and hydraulic conductivity as major physical parameters makes possible prediction of filter performance by conducting simple pilot plant tests using small, shallow experimental filters. The model provides an effective, rapid tool to be used in the generation of data for optimization techniques that are aimed at the improvement of filter productivity and to obtain better utilization of filter capacity.

2.6.3 Variables Affecting Filtration

The operation of rapid filters is affected by a number of variables. Some of these are determined at the design stage and others during operation. These variables are : (i) depth of media; (ii) grain size media; (iii) grain material; (iv) rate of filtration; (v) inflow concentration; (vi) type of suspension and (vii) water or wastewater temperature. Another variable is porosity but in practice it is a constant for a given grain material. Ives and Sholji (25) investigated some of these variables keeping grain material (Leighton Buzzard Sand) and suspension concentration (200 mg/l polyvinylchloride powder microspheres of diameter 1.3 micron) constant. The suspension was filtered through the sand of various uniform grain sizes, at various filtration rates from 0.6 to 2.0 $l/m^2.s$ (0.9 to 3.0 gpm/ft^2) and at various water temperatures from 3.5°C to 33°C. Some variables such as grain size distribution, composite beds of different media and direction of the flow were not examined because of their theoretical complexity. They concluded that the solids removal efficiency was inversely proportional to the filtration rate, the filter grain size, and the square of the viscosity of water. They compared their results with those of a number of previous theoretical and empirical studies and concluded that the general removal efficiency could be represented by :

$$\lambda \propto \frac{1}{v^{\alpha} d^{\beta} \mu^{r}}$$

where

λ = filter coefficient in model for removal with depth $\frac{\partial C}{\partial L} = -\lambda C$

C = particle concentration

L = depth in the filter bed

V = filtration rate

d = grain size

μ = water viscosity

$$\left.\begin{array}{l}\alpha = 1\\ \beta = 1\\ r = 2\end{array}\right\}\text{ By Ives and Sholji's experiments}$$

They concluded that the values of the exponents may vary with different suspensions. Various studies showed the greatest difference in the exponent β, ranging from 1 to 2.5, with several values around 1.7 for real filtration data.

2.6.4 Effect of Filtration Rate on Quality

A filtration rate of 1.4 $l/m^2.s$ (2.0 gpm/ft^2), for chemically pretreated surface waters, was considered virtually inviolable for the first half of the twentieth century in United States. However several workers have observed that with properly pretreated water, higher rates give the same effluent quality. Brown (26) compared performances at the filtration rates of 1.4, 2.0 and 2.7 $l/m^2.s$ (2.0, 3.0 and 4.0 gpm/ft^2) on full scale filters treating water that had already received pretreatment of alum coagulation and sedimentation. He observed that the difference in effluent turbidity and bacterial content was negligible. In a study on granular filters for tertiary wastewater treatment, Baumann and Huang (27) concluded that the high flow rate had no significant effect on filtrate quality. Also headloss development was found to be related to suspended solids accumulation within the filter pores and was relatively unaffected by filtration rate.

Baylis (28) reported the seven years testing on a full scale at Chicago. The performance of the filters was compared at 1.4, 2.7, 3.1 and 3.4 $l/m^2.s$ (2.0, 4.0, 4.5, and 5.0 gpm/ft^2). He concluded that the filtration rate of 3.4 $l/m^2.s$ (5.0 gpm/ft^2) did not depreciate the effluent quality, especially with regard to bacterial content. The pre-treatment involved alum coagulation with activated silica during times of weak flocculation. Baylis acknowledged that floc would pass through the filters even at 1.4 $l/m^2.s$ (2.0 gpm/ft^2) without the activated silica.

Robeck et al. (29) compared the performance of pilot filters at 1.4, 2.7, and 4.1 $l/m^2.s$ (2.0, 4.0, and 6.0 gpm/ft^2) filtering alum-coagulated surface water through single medium and dual-media filters. They concluded that with proper coagulation ahead of filters, the effluent turbidity, coliform bacteria, poliovirus, and powdered carbon removal was as good at 4.1 $l/m^2.s$ (6.0 gpm/ft^2) as at 2.7 or 1.4 $l/m^2.s$ (4.0 or 2.0 gpm/ft^2). The pretreatment included activated silica as flocculation aid, and a polyelectrolyte as a filter aid.

Although high filtration rates do not depreciate the effluent quality but according to recent studies, a large number of particulate may be present in the filtered effluent of low turbidity. Therefore, the policy of pushing filtration rates higher and higher should be avoided in order to produce high quality water at all times (23).

2.7 CHLORINATION

Although other bactericidal agents may be used to disinfect wastewater, chlorine is the only one that has widespread application. The purpose for chlorinating wastewater effluent is to protect public health by inactivating pathogenic organisms including enteric bacteria, viruses, and protozoans. Satisfactory disinfection of a secondary effluent is defined by an average fecal coliform count of less than 200 per 100 ml. Safety of effluent disposal by dilution in surface waters after chlorination is based on the argument that reduction of fecal coliforms from 10^6/100 ml to 200/100 ml eliminates the great majority of bacterial pathogens and inactivates large numbers of enteric viruses. Chlorine dosage needed for disinfection depends on wastewater pH, presence of interfering substances, temperature, and contact time. Applications of 8 to 15 mg/l provide adequate disinfection in well designed units with a minimum contact time of 20 to 30 minutes (30).

The destruction of pathogens by chlorination is dependent upon water temperature, pH, time of contact, degree of mixing, turbidity, presence of interfering substances, and concentration of chlorine available. When chlorine is dissolved in water at temperatures between 4.5 to 100°C it reacts to form hypochlorous and hypochloric acids :

$$Cl_2 + H_2O \longrightarrow HOCl + HCl$$

The hypochlorous acid ionizes practically instantaneously into hydrogen and hypochlorite ions :

$$HOCl \rightarrow H^+ + OCl^-$$

Chloramines formed in presence of ammonia are much less effective disinfectants than free chlorine. In the presence of ammonia, a three chloramines are obtained :

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad \text{(monochloramine)}$$

$$NH_3 + 2HOCl \rightarrow NHCl_2 + 2H_2O \quad \text{(dichloramine)}$$

$$NH_3 + 3HOCl \rightarrow NHCl_3 + 3H_2O \quad \text{(trichloramine)}$$

The formation of monochloramine and dichloramine depends on the relative concentrations of chlorine and ammonia as well as with pH and temperature. Above pH about 9.0, monochloramines exist almost exclusively; at pH about 6.5, monochloramines and dichloramines coexist in approximately equal amounts; below pH 6.5 dichloramines predominate; and trichloramines exist below pH about 4.5. The point where all ammonia is converted to trichloramine or oxidized to free nitrogen is referred to as the breakpoint. Chlorination below this level is combined while above this level is free available residual chlorination. Figure 2.4 illustrates the relationship between chlorine concentration and the contact time required for 99 percent destruction of E. Coli for three different forms of chlorine (15).

Table 2.2 summarizes a number of studies reporting inactivation data, using free or combined chlorine (31). In Pomona virus study (12), a 10 mg/l residual chlorine concentration was found enough to reduce coliform count to 2.2 per 100 ml.

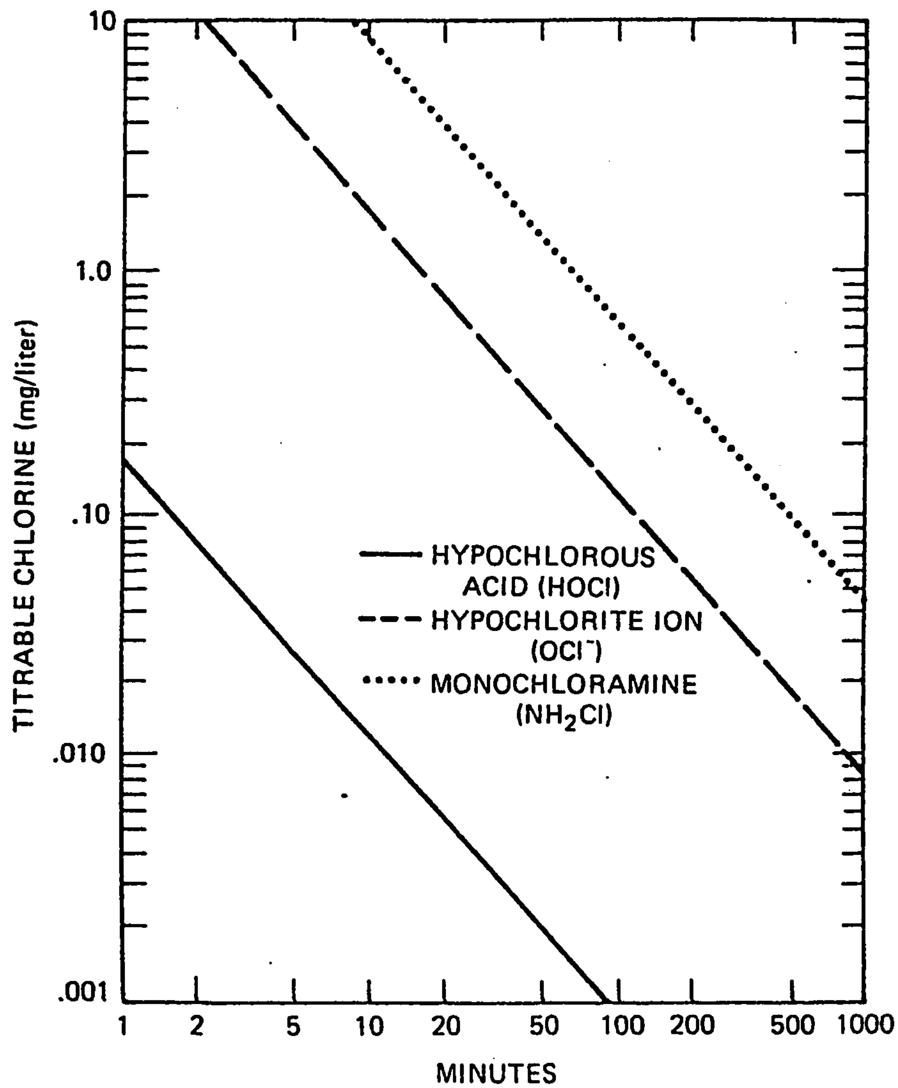

Fig. 2.4 : Relationship Between Concentration and Time for 99 Percent Destruction of E. Coli by Three Forms of Chlorine at 2-6°C (15)

Table 2.2 : Inactivation of Microorganisms by Chlorine

Organisms	Medium	Dose (D) or Residual (R) (mg/l)	contact Time (min)	Criteria
Fecal Coliform	Lagoon effluent	18-28(D)	-	99.99% inactivation
Fecal Coliform and fecal streptocci	Secondary effluent	0.10 (R)	-	99.9-99.99% inactivation
Fecal Coliforms	Blackwater	<750(D)	-	200/100 ml
Total viable Count	Secondary effluent	5(D)	15	99.0% inactivation
Bacteriophage T4	Advanced wastewater treatment effluent	0.4(R) free	26	99.9% inactivation

Chapter 3

MATERIALS AND METHODS

Selection of appropriate advanced wastewater treatment unit processes depends on the type of pollutant to be removed. Table 3.1 lists popular advanced treatment methods (22). Factors to be considered when designing an advance wastewater treatment facility are the disposition or reuse of final effluent, required effluent quality, nature of the materials to be removed, the problems associated with sludge handling, the potential for recovery and reuse of coagulants and other materials used in the treatment process, the demand for energy and other consumable resources, and overall economy.

3.1 DESCRIPTION OF THE PROCESS

The process employed for this study consisted coagulation, flocculation, sedimentation, granular media filtration and batch chlorination. The schematic diagram of the process is shown in Fig. 3.1. Coagulation and flocculation are the processes that follow sequentially. They are distinguished primarily by the types of chemicals used for their initiation and the size of the particles developed. Coagulation is the conversion of finely dispersed colloids into small floc on the addition of the coagulants like alum and iron salts including ferric chloride, ferric sulfate and ferrous sulfate. Flocculation is the agglomeration of small, slow settling floc formed during coagulation into larger floc that settle rapidly. Sedimentation process is used to remove the solid particles

Table 3.1 : Selected Advanced Wastewater Treatment Processes

S.No.	*Type of Impurity to be Removed*	*Unit Processes Employed*
1.	Suspended Solids	Filtration through granular beds. Microscreening. Chemical coagulation and clarification
2.	Organic Matter	Adsorption on granular activated carbon. Extended biological oxidation.
3.	Phosphorus	Biological-chemical precipitation and clarification. Chemical coagulation and clarification Irrigation of cropland.
4.	Nitrogen	Biological nitrification-denitrification. Ammonia reduction by air stripping. Breakpoint chlorination. Irrigation of cropland.
5.	Heavy Metals	Lime precipitation.
6.	Dissolved Solids	Reverse Osmosis.

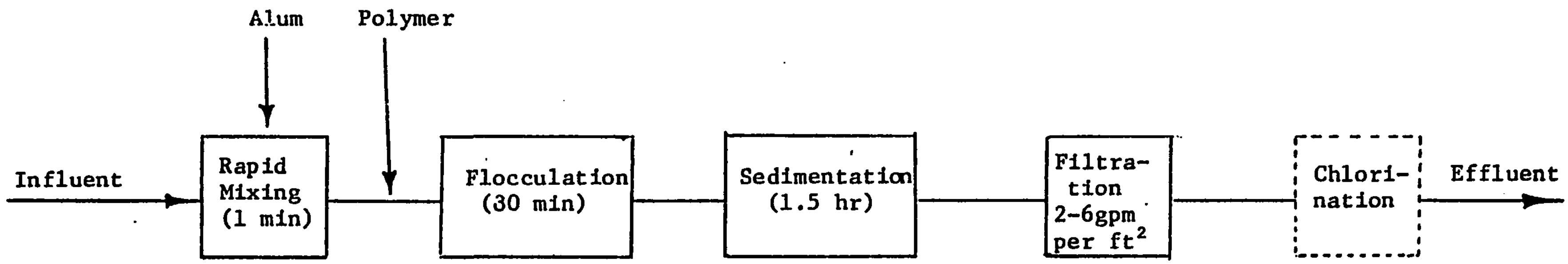

Figure 3.1: Flow Diagram for Tertiary Treatment of ARAMCO Wastewater Effluent

from suspension by gravity. Sedimentation following flocculation depends on the settling characteristics of the floc formed in the coagulation process (22). Filtration is the key process to remove particulate nonsettleable solids from water and wastewater by passing through a porous medium. Efficient filtration can increase the removal of suspended solids, turbidity, biochemical oxygen demand (BOD),chemical oxygen demand (COD), heavy metals, bacteria, viruses and other substances. Chlorination is the most common disinfection process to destroy or inactivate the pathogenic organisms.

The selection of the optimum type and dosage of coagulant cannot be made for any water or wastewater without experimentation. Point of application of chemicals is also a very important consideration. Coagulation takes place in a rapid mix basin. The primary function of the rapid mixing is to disperse the coagulant throughout the wastewater. Coagulant aid helps in bridging the floc to produce bigger and settleable floc. Polymer may be applied before the flocculation basin as a flocculation aid, or before the filter as filter aid. The chemicals used for coagulation in this study were alum and Magnafloc 155 anionic polymer. Their points of application are shown in Fig. 3.1.

The filter used for this study was a rapid sand gravity filter with the provision of backwashing. Rate of filtration varied from 1.4 to 4.1 l/m^2.sec (2 to 6 gpm/ft^2). The wastewater feeding was continuous at the rate of 0.48 m^3/day (i.e. 335 ml/min). Filter was backwashed after 8 hours of run. The rapid sand filter is not very effective for removal of bacteria, therefore subsequent chlorination is essential for protection of public health. For this study, batch chlorination was done on the filtered effluent.

3.2 DESCRIPTION OF THE EXPERIMENTAL APPARATUS

This study was conducted on a bench scale in a U.P.M laboratory. The system shown schematically in Fig. 3.1 includes the individual unit processes of rapid mixing, flocculation, sedimentation and filtration. The purpose of this section is to describe these unit processes.

3.2.1 Rapid Mix Tank

The rapid mix tank was used to thoroughly mix alum with the secondary effluent. The details of rapid mix tank are shown in Fig. 3.2. The tank was circular in plan section with an internal diameter of 7.2 cm. At a water depth of 11 cm, the tank provided a detention time of 1.3 min at a flow rate of 0.48 m^3/day (335 ml/min). A baffle wall of width 1.8 cm and thickness 0.5 cm was provided near the outlet to avoid vortices and create turbulence for proper mixing. The tank was equipped with a magnetic stirrer and a magnetic bar to provide continuous mixing. The alum coagulant and the wastewater were pumped into at the bottom of this tank directly from a chemical solution tank and wastewater holding tank respectively.

3.2.2 Flocculation Tank

The flocculation tank shown in Fig. 3.3 contained two baffles

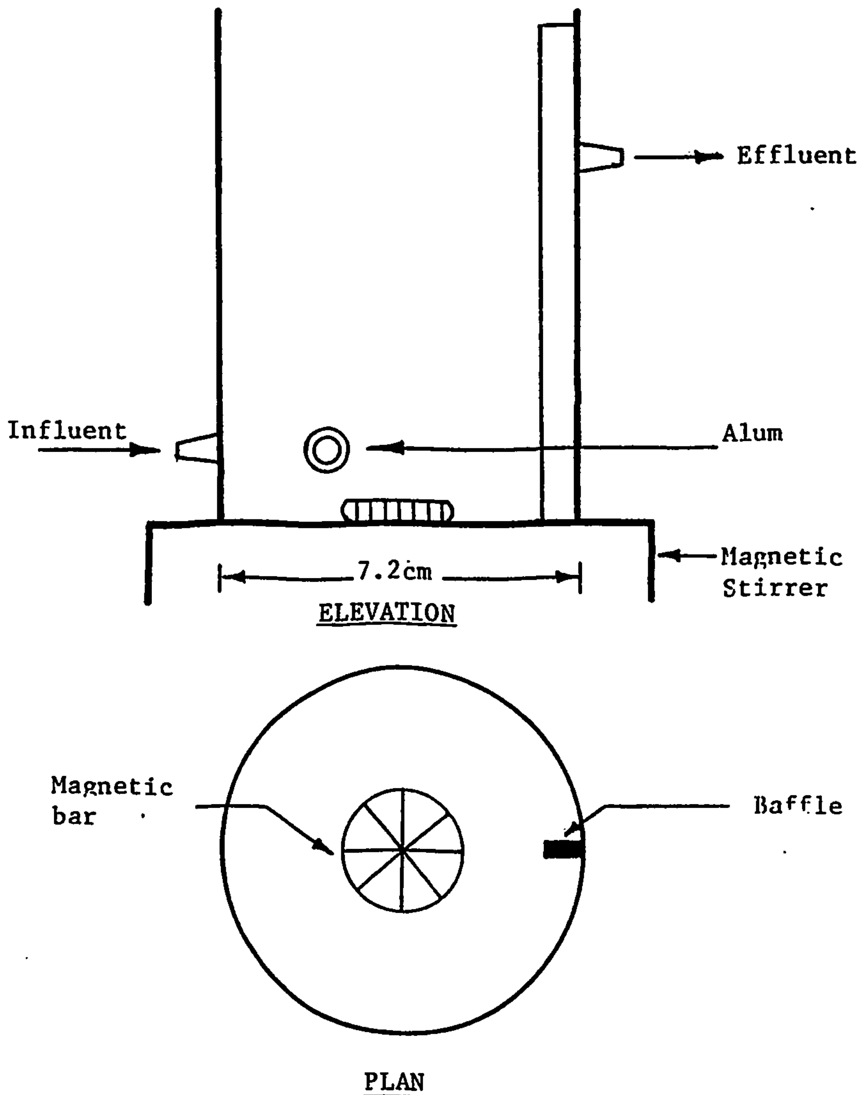

Fig. 3.2 : Details of Rapid Mix Tank

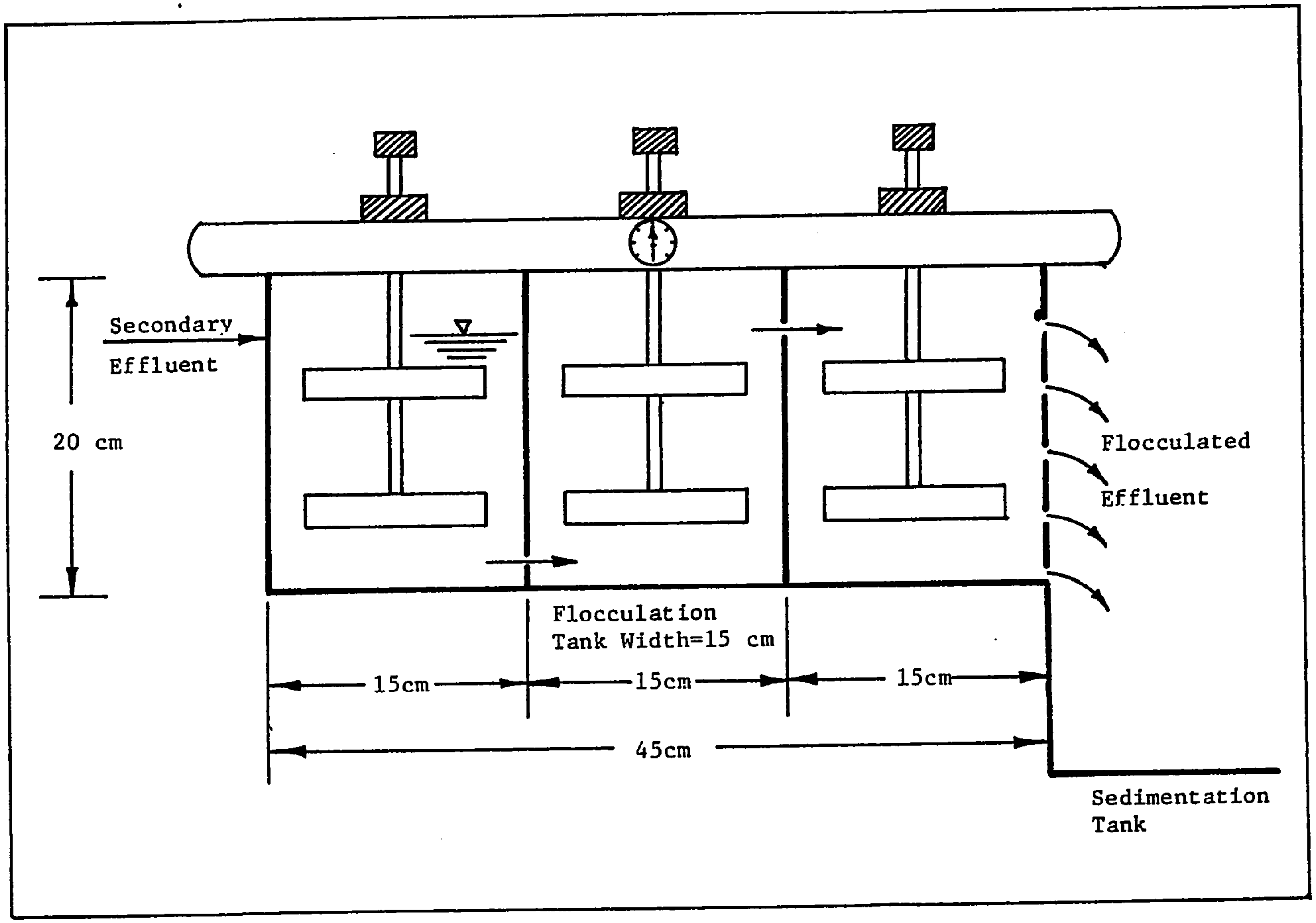

Fig. 3.3 : Details of Flocculation Tank

subdividing it into three compartments. The first baffle contained 5 holes of 1 cm diameter of 1.5 cm from the bottom and the other contained the same kind of holes at 12.5 cm from the bottom of the tank and thus providing an under-over flow pattern. The outlet of the flocculation tank, with 25 holes (5 holes in each line) of the same diameter, opened in the sedimentation tank. Each compartment of the flocculation tank was 15 cm x 15 cm in plan section and was equipped with horizontal paddle mixing device. The mixing device had a variable speed adjustment to provide a range of 0 to 100 rpm. At a water depth of 15.6 cm, the flocculation tank provided a total detention time of 31 minutes at a flow rate of 335 ml/min. The Magnafloc 155 anionic polymer used as a coagulant aid was injected between the rapid mix tank and flocculation tank into the influent line. Design of the flocculation basin is given in Appendix A.

3.2.3 Sedimentation Tank

The sedimentation tank shown in Fig. 3.4 was 65 cm long, 15 cm wide, and 34 cm deep. At a water depth of 30.6 cm, the tank provided a total detention time of 89 min at a flow rate of 335 ml/min and an overflow rate of approximately 5.0 $m^3/m^2.d$. The sedimentation tank was attached to the flocculation tank.

3.2.4 Granular Media Filter

The filter used for this system was a granular media sand filter. Details of the filter are shown in Fig. 3.5. The filter had a diameter

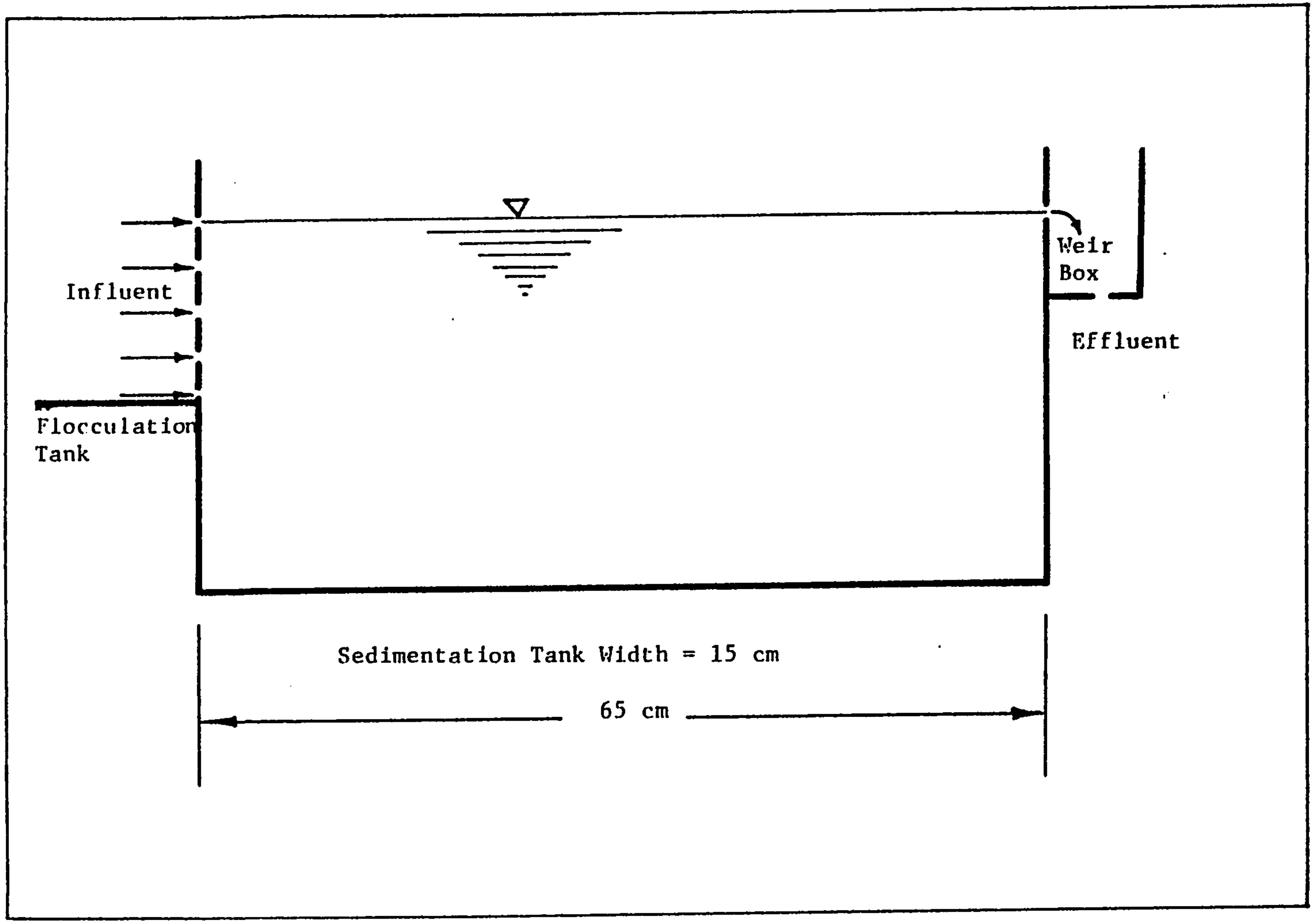

Fig. 3.4 : Details of Sedimentation Tank

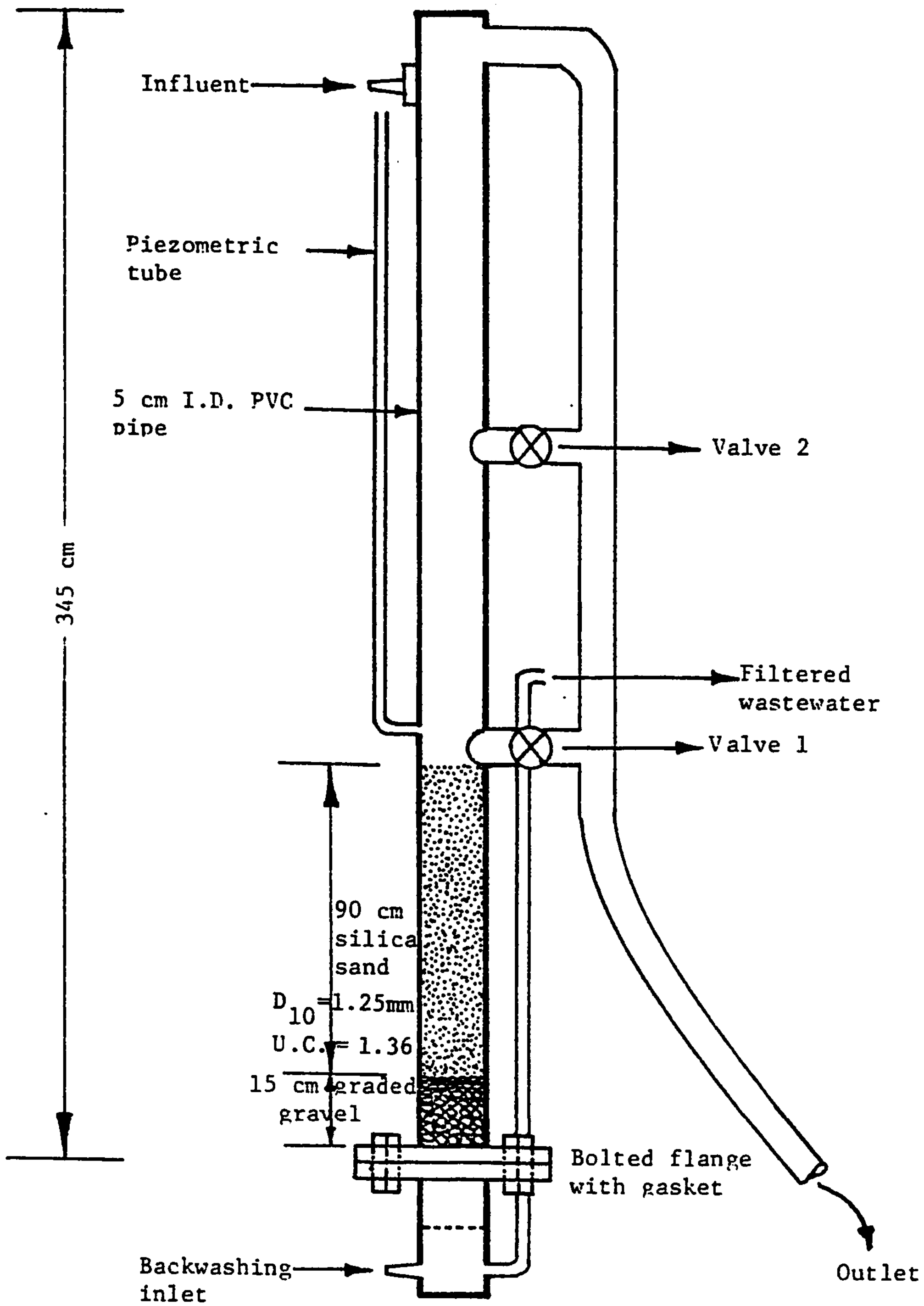

Fig. 3.5 : Details of Granular Media Filter

of 5.0 cm and a total depth of 345 cm. The 90 cm deep sand layer with an effective particle size of 1.25 mm and uniformity coefficient of 1.36 overlaid a 15 cm deep graded gravel underdrain system with effective particle size of 2.40 mm and uniformity coefficient of 1.92. Filter was provided with the backwash facilities.

3.2.5 General Accessories

Various other accessories were used to support the performance of the system. The holding tank for wastewater had a capacity of about 150 litre. The pumps used to feed the wastewater and alum into the rapid mix tank were Masterflex model No. 7015 and 7014 respectively. Masterflex model No. 7014 and 7017 pumps were used for feeding polymer on line and clarified wastewater to the sand filter. Hatch turbidimeter model 2100A was used to determine the turbidity of samples.

3.3 WASTEWATER

Treatability of two kinds of wastewater was studied. Unchlorinated secondary wastewater effluent from the final clarifiers and chlorinated secondary effluent from wetwell of North Aramco Wastewater Treatment Plant, Dhahran, were obtained for this study. Fig. 3.6 shows the two locations where from the above samples were collected. This plant has a capacity of treating 8.0 million gallons of wastewater per day utilizing extended aeration mode. The samples were collected daily around 3.00 p.m. and used next morning. Table 3.2 lists the

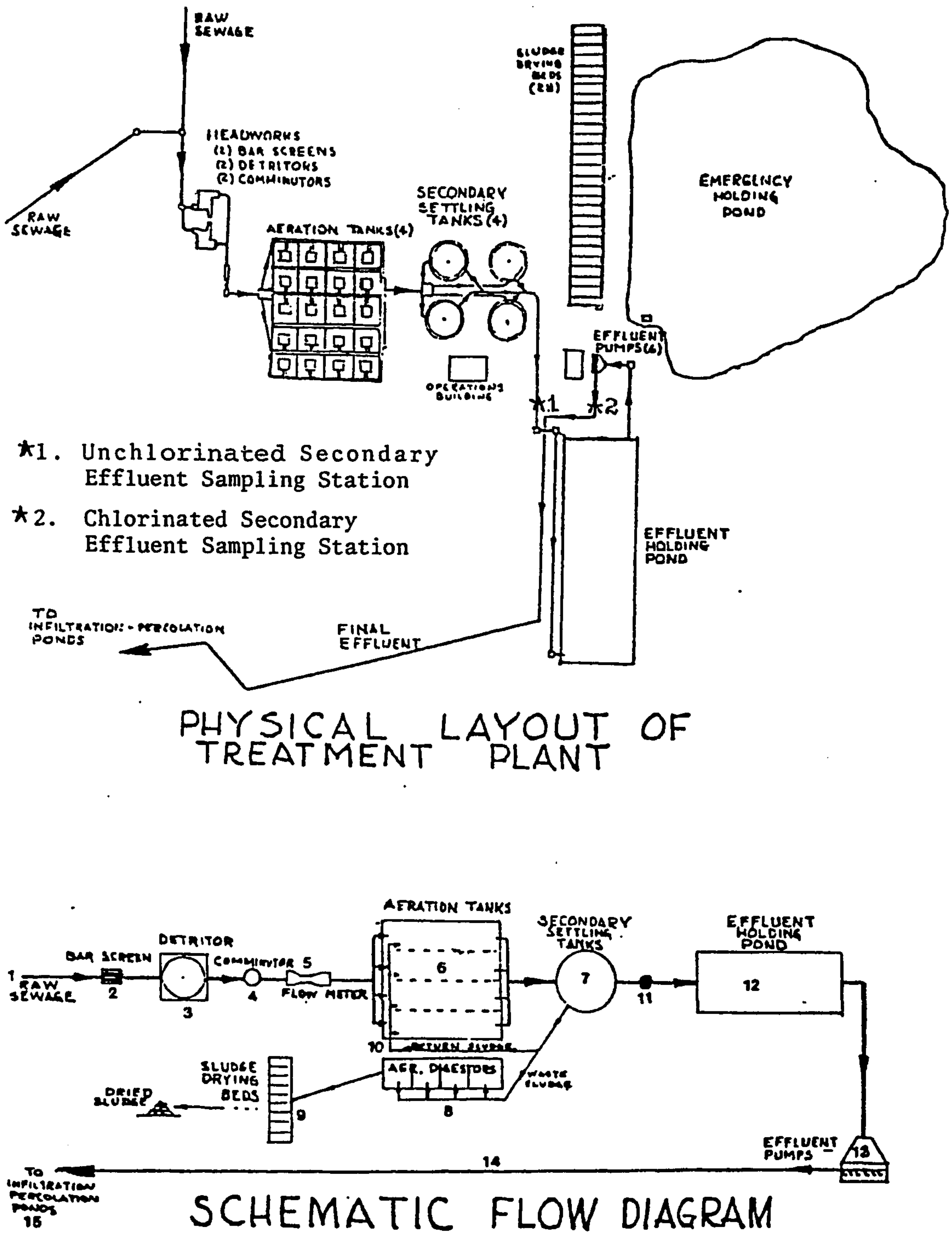

Fig. 3.6 : North Aramco Wastewater Treatment Plant, Dhahran

Table 3.2 : Secondary Wastewater Effluent Characteristics During Summer 1985

Parameter	*Unchlorinated Secondary Effluent*	*Chlorinated Secondary Effluent*
pH value	7.8	7.9
Temperature, °C	28	30
Biochemical oxygen demand, mg/l	<5	-
Total suspended solids, mg/l	<5	<5
Total dissolved solids, mg/l	3600	3600
Residual chlorine, mg/l	-	2.0
Coliform concentration per 100 ml	150,000	-

various characteristics of the wastewater during Summer 1985.

3.4 PRELIMINARY STUDIES

The preliminary phase of the studies was to hydraulically define the treatment apparatus and determine preliminary chemical dosages.

3.4.1 Tracer Study

The hydraulic character of a reactor is defined by the dye-tracer response curve which is the residence time distribution of individual particles of liquid flowing through the tank. The theoretical hydraulic detention time is defined as the volume of the reactor divided by the rate of flow through the reactor. A tracer study was performed on the flocculation tank to determine experimental detention time and to ensure the proper mixing of chemicals. For making test, a pulse input of 10 ml rhodamine dye having concentration 1000 mg/l was applied at the inlet of the flocculation tank. Effluent samples were collected at different interval of time and their dye concentrations were determined using a spectrophotometer. A standard curve for rhodamine dye was prepared earlier to determine the effluent concentration. Rapid mix tank was very small with too short a detention time to perform a tracer study. Therefore, mixing was observed by injecting a small amount of dye and observing its dispersion visually.

3.4.2 Jar Tests

The purpose of the jar tests was to study chemical coagulation and flocculation for selecting primary chemical dosages for running the bench scale unit. Various chemical dosages were applied to the wastewater effluent and formation of floc, their size and settleability were observed. For making the jar test 1 litre of wastewater was placed in each of six beakers. The jars were dosed with different amounts of alum. After rapid mixing at a speed of about 100 rpm for 1 minute, Magnafloc 155 anionic polymer was added. Then the jars were stirred at a speed of 45 rpm for 30 min. After stopping the stirrer, the nature and settling characteristics of the floc were observed and recorded.

3.5 EXPERIMENTAL PROCEDURE

This study was performed on the bench scale unit as shown in Fig. 3.7. The first series of the tests treated unchlorinated secondary wastewater effluent and the second series treated the chlorinated secondary effluent, both from North ARAMCO Wastewater Treatment Plant, Dhahran. Following are the details of the individual process operation.

3.5.1 Rapid Mixing, Flocculation and Sedimentation

Based on a continuous feed of 335 ml of wastewater per minute (0.48 cubic metre/day) by pump#1 , the rapid mixing was done for 1 minute, flocculation 30 minutes, and sedimentation 1.5 hours. Alum, used as coagulant, was fed into the rapid mix tank directly from a chemical solution tank using a Masterflex model No.7014 pump(pump#2). A similar type of pump (pump#3) was used to inject Magnafloc 155

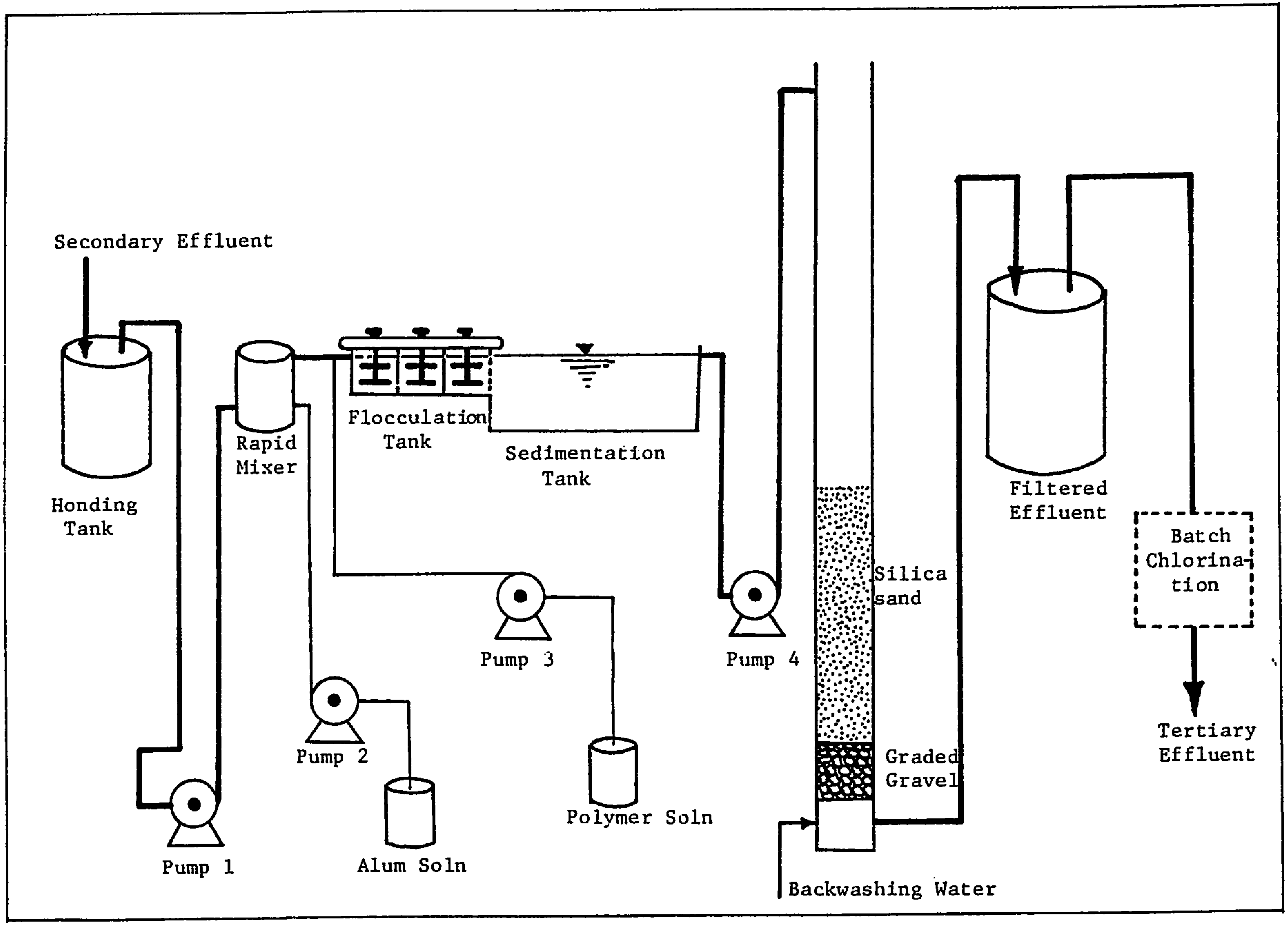

Fig. 3.7 : Schematic Diagram of Laboratory Apparatus

anionic polymer into the line between the rapid mix and flocculation tank. The total time of operation for rapid mix, flocculation and sedimentation tank was more than 10 hrs per day.

3.5.2 Granular Media Filtration

The sand filter was used to remove the suspended solids not settled in the sedimentation tank. The filter was operated at hydraulic loading of 2.7 $l/m^2.s$ (4 gpm/ft^2) 8 hours per day for different chemical dosages. The turbidities of influent, settled, and filtered effluent and the filter head loss were recorded in the beginning at a time interval of half-an-hour and later at 1 hour time interval. The average initial head loss recorded after 5 minutes of starting the filter was found to be approximately 17 cm. The filter was backwashed every day after each filter run at the rate of 3800 ml/min (31.2 $l/m^2.s$) using tap water available in the laboratory for about 5 minutes. The filter hydraulic loading was changed to 1.4 and 4.1 $l/m^2.s$ (2 and 6 gpm/ft^2) for a particular chemical dosage to see the effect of hydraulic loading on effluent filtered turbidity and filter head loss for both chlorinated and unchlorinated secondary wastewater effluents. The purpose of the whole study was to determine the best operation based on turbidity in the filtered wastewater and head loss developed in 8 hr of filter run.

3.5.3 Batch Chlorination

Portions of the filtered unchlorinated secondary effluent were chlorinated at various dosage to find the contact time and chlorine residual necessary to reduce the coliform concentration to 2.2 per 100 ml. For making the test, a 2 litre filtered effluent was taken in a closed bottle and a selected chlorine dosage was applied. At the time of application, the wastewater was shaken thoroughly for 1 minute to mix the chlorine solution properly with wastewater. Residual chlorine was measured after 10, 20, 40 and 80 minute by DPD Ferrous Titriometric Method as described in Standard Methods (32). At the same time, sample portions were collected in already sterilized bottles which contained 5 ml of 0.1N sodium thiosulphate solution, that stops further disinfection, to determine coliform counts. The process was repeated for all unchlorinated filtered secondary effluent samples treated at different chemical dosages.

3.5.4 Total Coliform Detection

The membrane filter coliform technique was used for the detection of coliforms. Fig. 3.8 shows the diagram of this technique. The apparatus to perform membrane filter coliform testing included a filtration unit, sterile filter membranes with a pore opening of 0.45 micrometer, culture dishes, Endo-agar as nutrient media, and forceps. Before each use, the forceps were sterilized. Decontamination of the filter units between successive filtrations was accomplished by exposing it to a flame for few seconds. A 100-ml sample of collected wastewater after chlorination process was filtered through the membrane. Using the forcep, the membrane was removed from the filtration unit and placed on already prepared endo-agar culture dish. After incubation for 24 hrs

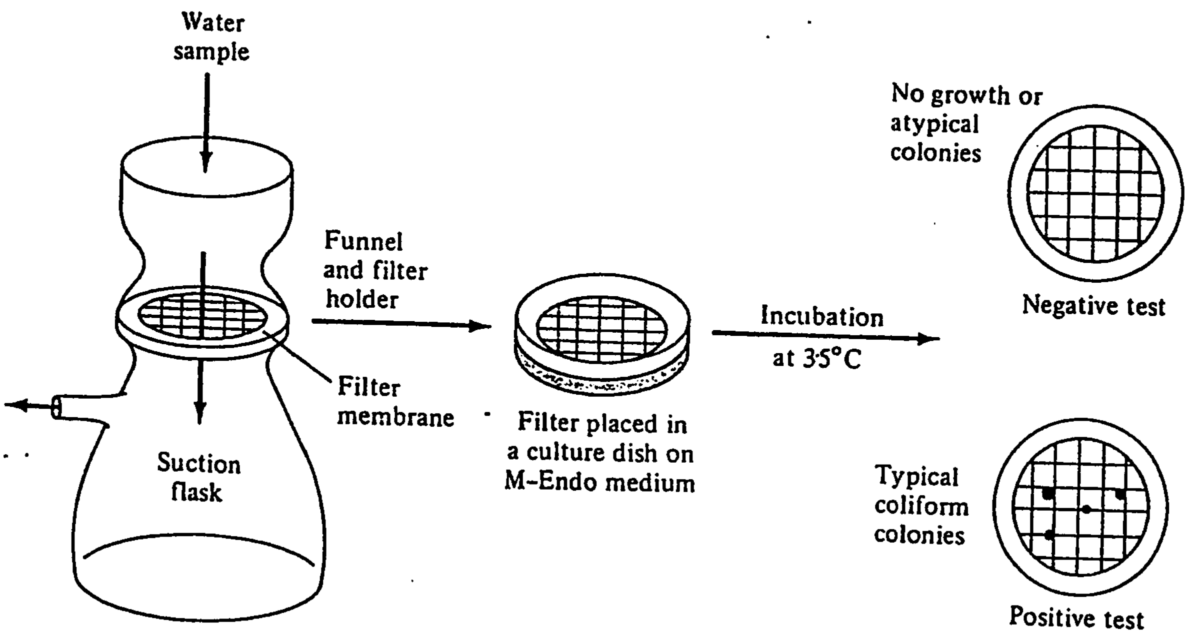

Fig. 3.8 : Diagram of Membrane Filter Technique for Coliform Testing (22)

at 35°C, the filter membranes were examined for coliform growth. Coliform colonies with pink to dark-red color with sometimes a green metallic surface sheen were counted. Counting was sometimes difficult because of atypical heavy growth of coliform colonies.

3.5.5 Analytical Techniques

Almost all analyses were carried out according to the Standard Methods (32) with slight modifications where needed. Table 3.3 gives the analytical techniques involved for different type of analyses.

Table 3.3 : Procedure for Water Quality Analysis

Types of Analysis	*Analytical Procedure*
pH	Standard Method
Turbidity	Hach Turbidimeter, Model 2100
Biochemical Oxygen demand	Standard Method
Suspended Solids	Standard Method
Total Dissolved Solids	Standard Method
Chlorine Residual	DPD ferrous titriometric method - Standard Method
Total Coliform	Membrane filter coliform technic - Standard Method

Chapter 4

EXPERIMENTAL RESULTS

Preliminary studies included jar tests and a tracer study on the flocculation tank. The whole bench scale study was performed for particular chemical dosages obtained suitable from the jar tests. Two kinds of wastewater effluent from ARAMCO Wastewater Treatment Plants were given tertiary treatment by sedimentation and sand filtration. The purpose of this chapter is to present the results of experiments conducted.

4.1 JAR TEST RESULTS

Table 4.1 and Table 4.2 contain the jar-test results for unchlorinated and chlorinated secondary effluent respectively. Tests were repeated two or three times to check the reliability of results. Settleable floc developed for an alum dosage between 5 and 20 mg/l and polymer in the range 0.1 to 0.3 mg/l. An alum dosage above 20 mg/l did not help coagulation in any case. Also, alum alone was not sufficient for coagulation and a dosage below 5 mg/l alum was insufficient for coagulation along with any dosage of polymer. Therefore, the chemical dosages chosen for operation of the whole system were alum as 5 to 20 mg/l and polymer as 0.1 to 0.3 mg/l.

4.2 RESULTS OF TRACER STUDY

Table 4.3 lists the data obtained from tracer study on the

Table 4.1 : Results of Jar Test for Unchlorinated Secondary Effluent

Test Series	*Case No.*	*Chemical Dosage (mg/l) Alum*	*Polymer*	*Observations*
A	1	5	0.1	Large flocs, Settleable, Very clear effluent
	2	10	0.1	Smaller than case # 1, Settleable, Clear effluent
	3	15	0.1	Very small flocs, Suspended, Not very clear effluent
	4	20	0.1	Same as Case # 3
	5	30	0.1	Less and very small flocs, Suspended, Not clear effluent
	6	40	0.1	Same as Case # 5
B	1	5	0.2	Few big flocs, Settleable, Very clear effluent
	2	10	0.2	Many large flocs, Settleable, Very clear effluent
	3	15	0.2	Small flocs, Partially settleable, Clear effluent
	4	20	0.2	Same as Case # 3
	5	30	0.2	Less and small flocs, Suspended, Not clear effluent
	6	40	0.2	Same as Case # 5
C	1	5	0.3	Few and very large flocs, Settleable, Very clear effluent
	2	10	0.3	Many large flocs, Settleable, Very clear effluent
	3	15	0.3	Many small flocs, Partially settleable, Clear effluent
	4	20	0.3	Same as Case # 3
	5	30	0.3	Very small flocs, Suspended, Not clear effluent
	6	40	0.3	Same as Case # 5

Table 4.2 : Results of Jar Test for Chlorinated Secondary Effluent

Test Series	*Case No.*	*Chemical Dosage (mg/l) Alum*	*Polymer*	*Observations*
A	1	10	0.1	Many small flocs, Settleable, Clear effluent
	2	20	0.1	Many small flocs, Settleable, Not very clear effluent
	3	30	0.1	Smaller than case # 2, Settleable, Not clear effluent
	4	40	0.1	Small flocs, Suspended, Not clear effluent
	5	60	0.1	Same as Case # 4
B	1	5	0.2	Large flocs, Settleable, Clear effluent
	2	10	0.2	Many large flocs, Settleable, Very clear effluent
	3	20	0.2	Many small flocs, Settleable, Clear effluent
	4	30	0.2	Small flocs, Partially settleable, Not very clear effluent
	5	40	0.2	Same as Case # 4
	6	60	0.2	Very-very small flocs, Suspended, Not clear effluent
C	1	5	0.3	Large flocs, Settleable, Clear effluent
	2	10	0.3	Many flocs smaller than Case # 1, Settleable, Clear effluent
	3	20	0.3	Small flocs, Settleable, Not very clear effluent
	4	30	0.3	Same as Case # 3
	5	40	0.3	Very-very small flocs, Suspended, Not clear effluent
D	1	2	0.1	Few small flocs, Not settleable, Not clear effluent
	2	6	0.2	Large flocs, Settleable, Clear effluent
	3	10	0.3	Many flocs smaller than Case # 2, Settleable, Clear effluent
	4	14	0.4	Many small flocs, Partially settleable, Not very clear effluent
	5	18	0.5	More flocs than Case # 4, Settleable, Clear effluent

Table 4.3 : Data Obtained from Tracer Study on Flocculation Tank

Time (*min*)	*Output Concentration* (*mg/l*)	*Time* (*min*)	*Output Concentration* (*mg/l*)	*Time* (*min*)	*Output Concentration* (*mg/l*)
0	0.0	16.0	0.92	34.0	0.56
1.0	0.06	17.0	0.92	36.0	0.51
2.0	0.15	18.0	0.88	38.0	0.47
3.0	0.24	19.0	0.83	40.0	0.42
4.0	0.33	20.0	0.85	42.0	0.42
5.0	0.45	21.0	0.83	44.0	0.38
6.0	0.51	22.0	0.83	46.0	0.38
7.0	0.63	23.0	0.81	48.0	0.31
8.0	0.67	24.0	0.79	50.0	0.31
9.0	0.74	25.0	0.74	55.0	0.24
10.0	0.79	26.0	0.69	60.0	0.20
11.0	0.85	27.0	0.69	65.0	0.18
12.0	0.92	28.0	0.67	70.0	0.13
13.0	0.92	29.0	0.63	75.0	0.11
14.0	0.92	30.0	0.63	80.0	0.08
15.0	0.97	32.0	0.62	90.0	0.06

flocculation tank. This study was performed to ensure the proper mixing of chemicals. Fig. 4.1 was used to find the output concentration of the dye. The study was done for three detention times. Fig. 4.2 shows the output tracer distribution curve. The peak concentration was reached after 15 minutes. Table 4.4 gives the data for calculating the dispersion number as follows; the theoretical mean residence time t_R equals the volume of the flocculation tank divided by the flow

$$t_R = \frac{V}{Q} = \frac{8780\text{ml}}{335\text{ ml/min}} = 26.2\text{ min}$$

The amount of Rhodamine B dye injected in the influent was 10.0 mg in 10.0 ml of dye solution. Therefore, the calculated initial concentration based the volume of the flocculation tank is

$$C_o = \frac{10.0\text{mg}}{8.78\ \ell} = 1.14\text{ mg/l}$$

The variance σ^2 of a curve is calculated from a finite number of measurements i at equal time intervals by

$$\sigma^2 = \frac{\Sigma\, t_i C_i}{\Sigma C_i} - \left(\frac{\Sigma\, t_i C_i}{\Sigma C_i}\right)^2$$

$$\sigma^2 = \frac{9642}{7.20} - \left(\frac{219}{7.20}\right)^2 = 414$$

The mean residence time is

$$t = \frac{\Sigma\, t_i C_i}{\Sigma C_i}$$

$$t = \frac{219}{7.20} = 30.4\text{ min}$$

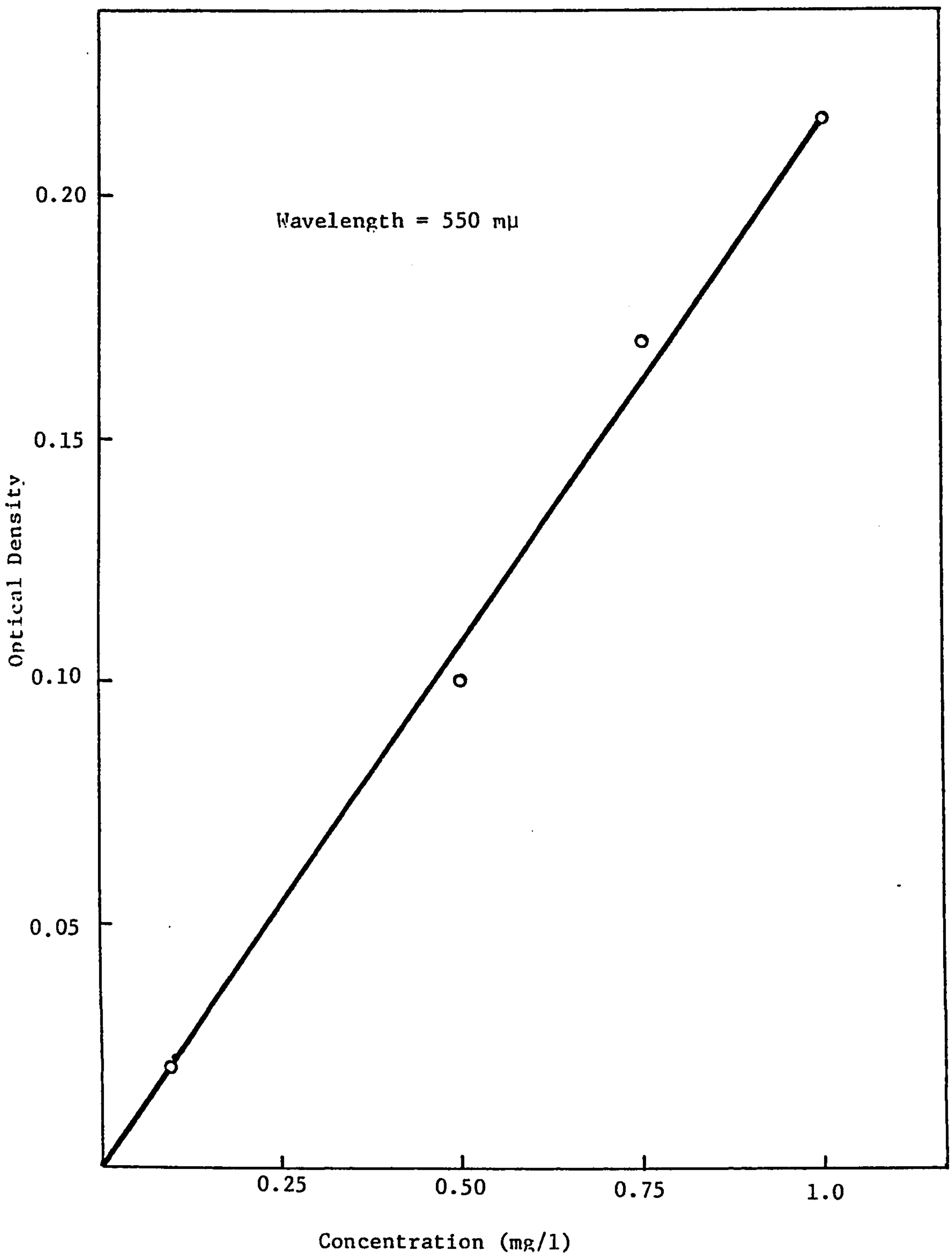

Fig. 4.1 : Optical Density Versus Concentration for Rhodamine-B Solution

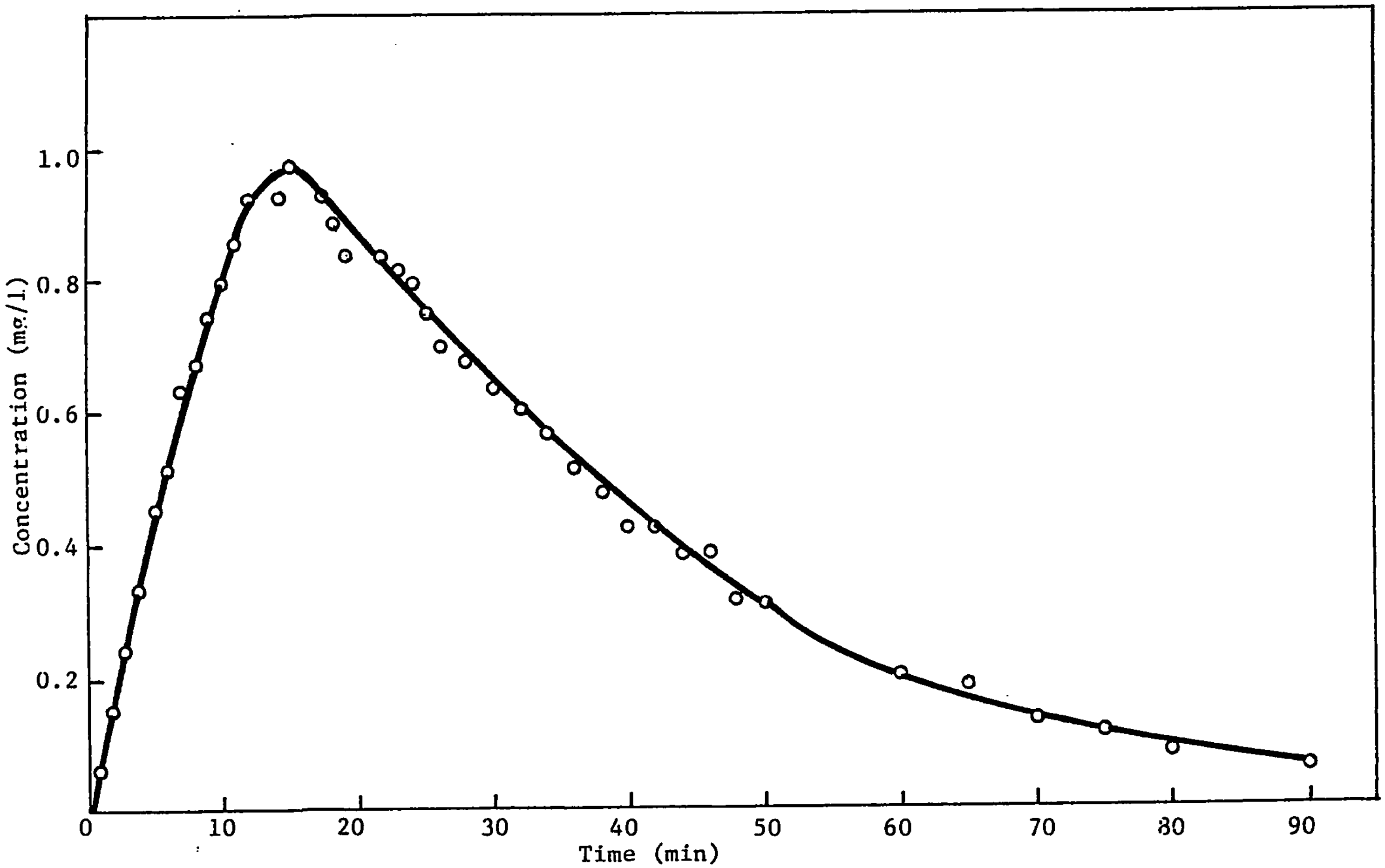

Fig. 4.2 : Output Tracer Distribution Curve for Flocculation Tank

Table 4.4 : Data for Calculating Dispersion Number

t_i (min)	C_i (mg/l)	t/t_R	C/C_o	t_iC_i	$t_i^2C_i$
5.0	0.43	0.19	0.38	2	11
10.0	0.80	0.38	0.70	8	80
15.0	0.96	0.57	0.84	14	216
20.0	0.87	0.76	0.76	17	348
25.0	0.74	0.95	0.65	19	463
30.0	0.64	1.15	0.56	19	576
35.0	0.55	1.34	0.48	19	674
40.0	0.46	1.53	0.40	18	736
45.0	0.38	1.72	0.33	17	770
50.0	0.30	1.91	0.26	15	750
55.0	0.24	2.10	0.21	13	726
60.0	0.20	2.29	0.18	12	720
65.0	0.16	2.48	0.14	10	676
70.0	0.13	2.67	0.11	9	637
75.0	0.11	2.86	0.10	8	619
80.0	0.09	3.05	0.08	7	576
85.0	0.08	3.24	0.07	7	578
90.0	0.06	3.44	0.05	5	486
SUM	7.20			219	9642

Thus the normalized variance is calculated as

$$\sigma_\theta^2 = \frac{\sigma^2}{\bar{t}^2} = \frac{414}{(30.4)^2} = 0.45$$

Finally, the dispersion number D/uL is determined by the relationship

$$\sigma_\theta^2 = 2\frac{D}{uL} - 2\left(\frac{D}{uL}\right)^2 (1-e^{-uL/D})$$

For $\sigma_\theta^2 = 0.45$, $D/uL = 0.34$

The comparison of this value with the values given in Fig. 4.3, yields a large amount of dispersion. Hence, the flocculation tank provided adequate mixing of chemicals. A computer program was run to obtain the data for plotting the theoretical effluent response to a pulse input (See Appendix B). Fig. 4.4 shows that experimental and theoretical effluent responses to a pulse input are comparable.

4.3 PERFORMANCE OF TREATMENT SYSTEM FOR UNCHLORINATED SECONDARY EFFLUENT

The treatment system (Fig. 3.1) was operated at various chemical dosages each for a period of 8 hours. Table 4.5 lists the experimental data obtained for unchlorinated secondary effluent at filter hydraulic loading of 2.7 $l/m^2.s$ (4.0 gpm/ft^2). Two or three runs were performed at each chemical dosage to check the consistency of results. The maximum influent turbidity was less than 2.0 nephlometric turbidity unit (NTU) while the minimum filtered effluent turbidity was 0.20 NTU. Head loss was very high when the dosage for polymer was increased. Table 4.6 lists the data obtained when the filter hydraulic loading was

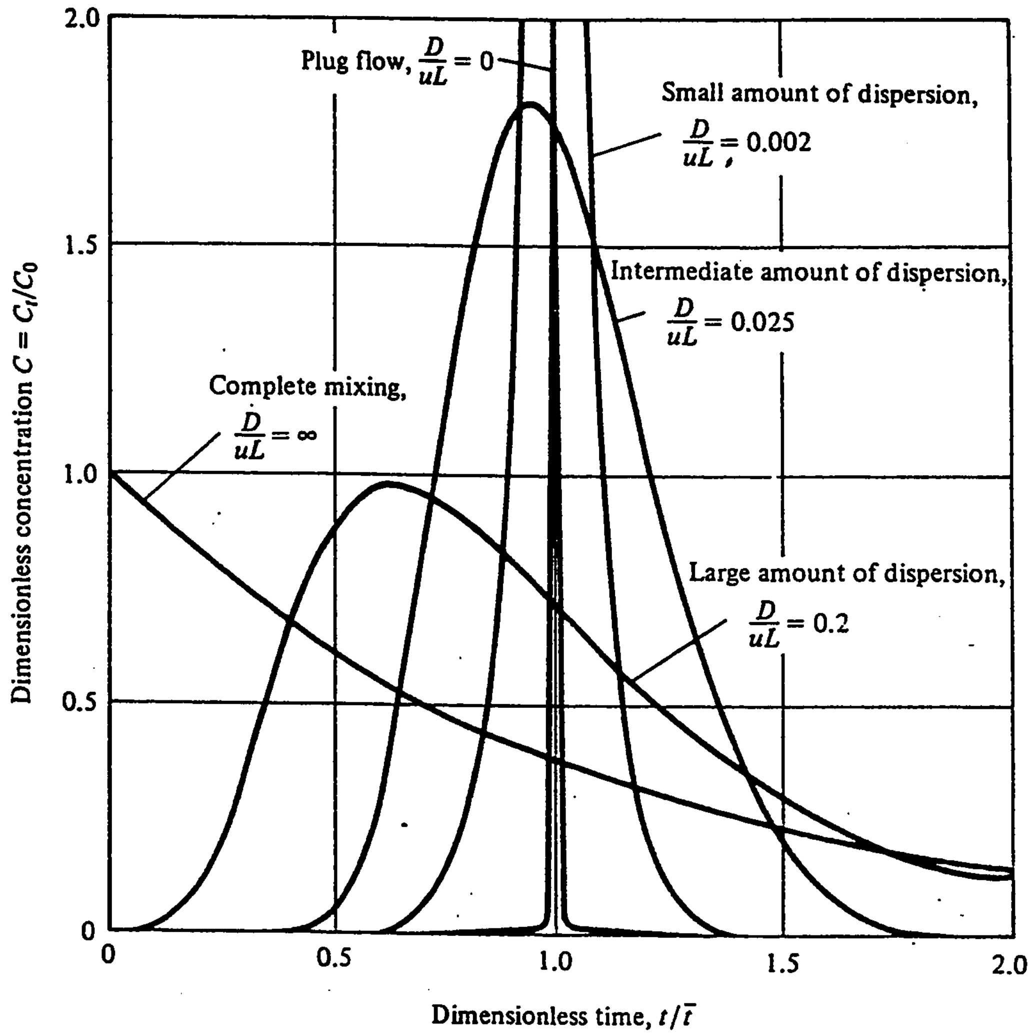

Fig. 4.3 : Normalized Residence Time Distributions in Response to a Pulse Input and Corresponding Dispersion Numbers for Various Degrees of Longitudinal Dispersion (22)

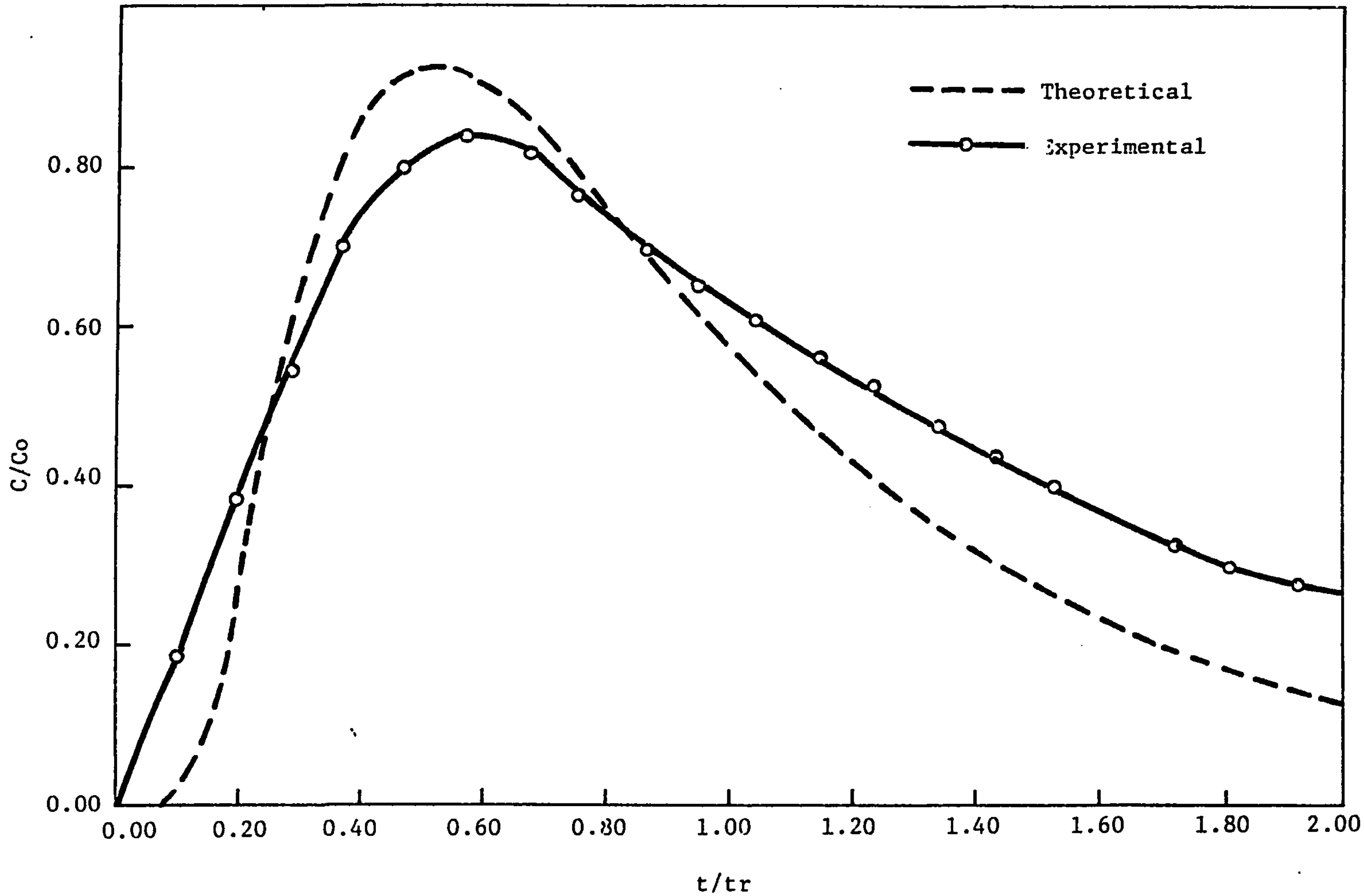

Fig. 4.4 : Normalized Effluent Response Curves for a Pulse Input

Table 4.5 : Experimental Data Obtained for Unchlorinated Secondary Effluent [Filter Loading = 2.7 $l/m^2.s$ (4.0 gpm/ft^2)].

Test Ser.	No. of Runs	Chemical dosage (mg/l) Alum	Polymer	Time (hr)	Avg. Turbidity (NTU) Influent	Settled	Filtered	Avg. Head loss (cm)
1	2	5.0	0.10	0.0	1.15	0.97	0.63	0.0
				0.5	1.15	0.90	0.49	4.1
				1.0	1.10	0.81	0.47	5.1
				1.5	1.30	0.80	0.44	6.1
				2.0	1.10	0.80	0.40	7.2
				3.0	1.10	0.72	0.28	10.0
				4.0	1.10	0.61	0.23	13.1
				5.0	1.05	0.59	0.21	16.2
				6.0	1.05	0.59	0.19	22.1
				7.0	1.05	0.59	0.19	28.1
				8.0	1.05	0.59	0.19	30.1
2	2	10.0	0.1	0.0	1.20	1.00	0.38	0.0
				0.5	1.15	0.92	0.24	3.5
				1.0	1.05	0.88	0.25	4.9
				1.5	1.05	0.87	0.24	5.5
				2.0	1.15	0.82	0.22	6.7
				3.0	1.10	0.79	0.22	8.8
				4.0	1.05	0.80	0.21	11.3
				5.0	1.05	0.81	0.20	13.7
				6.0	1.00	0.79	0.20	15.9
				7.0	1.05	0.78	0.21	18.3
				8.0	1.05	0.78	0.20	20.4
3	2	5.0	0.20	0.0	1.30	0.88	0.35	0.0
				0.5	1.30	0.79	0.33	2.2
				1.0	1.30	0.77	0.32	3.4
				1.5	1.20	0.72	0.31	4.9
				2.0	1.20	0.72	0.31	5.8
				3.0	1.20	0.65	0.30	8.6
				4.0	1.15	0.66	0.30	12.9
				5.0	1.10	0.63	0.29	18.2
				6.0	1.10	0.60	0.28	24.6
				7.0	1.10	0.61	0.28	30.6
				8.0	1.10	0.61	0.29	38.5

*Cont.

4	3	10.0	0.20	0.0	1.15	0.74	0.33	0.0
				0.5	1.15	0.74	0.27	3.3
				1.0	1.15	0.65	0.27	4.7
				1.5	1.10	0.63	0.26	6.4
				2.0	1.10	0.59	0.25	8.8
				3.0	1.10	0.54	0.25	12.4
				4.0	1.10	0.54	0.25	16.6
				5.0	1.10	0.61	0.24	21.9
				6.0	1.10	0.55	0.25	27.0
				7.0	1.75*	0.67	0.25	32.8
				8.0	1.75	0.67	0.25	38.3
5	2	5.0	0.3	0.0	1.70	0.81	0.42	0.0
				0.5	1.70	0.80	0.39	0.9
				1.0	1.70	0.77	0.38	1.4
				1.5	1.70	0.77	0.37	2.0
				2.0	1.70	0.77	0.37	2.9
				3.0	1.70	0.71	0.34	4.6
				4.0	1.65	0.65	0.31	6.9
				5.0	1.65	0.62	0.30	9.8
				6.0	1.60	0.59	0.30	14.4
				7.0	1.60	0.57	0.30	21.4
				8.0	1.60	0.57	0.30	27.6
6	2	10.0	0.3	0.0	1.30	0.59	0.39	0.0
				0.5	1.25	0.55	0.25	1.8
				1.0	1.15	0.51	0.25	4.1
				1.5	1.15	0.45	0.25	6.4
				2.0	1.15	0.48	0.25	10.6
				3.0	1.15	0.44	0.25	25.7
				4.0	1.10	0.43	0.25	38.4
				5.0	1.10	0.40	0.24	57.4
				6.0	1.10	0.40	0.23	79.4
				7.0	1.10	0.41	0.23	92.1
				8.0	1.10	0.41	0.23	107.8

* Change in the influent turbidity resulted from feed of a new sample of wastewater.

Table 4.6 : Experimental Data Obtained for Unchlorinated Secondary Effluent with varying Filter Hydraulic Loadings

Test Ser.	No. of Runs	Chemical dosage (mg/l) Alum	Chemical dosage (mg/l) Polymer	Time (hr)	Avg. Turbidity (NTU) Influ-ent	Avg. Turbidity (NTU) Settl-ed	Avg. Turbidity (NTU) Filter-ed	Avg. Head loss (cm)
1*	2	10.0	0.2	0.0	1.10	0.68	0.22	0.0
				0.5	1.10	0.67	0.23	0.8
				1.0	1.10	0.66	0.23	1.1
				1.5	1.10	0.65	0.24	1.3
				2.0	1.10	0.62	0.25	1.5
				3.0	1.10	0.62	0.25	2.1
				4.0	1.10	0.54	0.25	2.4
				5.0	1.10	0.55	0.25	2.9
				6.0	1.10	0.55	0.25	3.6
				7.0	1.45 *	0.55	0.25	4.2
				8.0	1.45	0.55	0.25	4.8
2*	2	10.0	0.2	0.0	1.50	0.55	0.31	0.0
				0.5	1.50	0.54	0.29	3.7
				1.0	1.50	0.54	0.29	3.7
				1.5	1.65	0.51	0.25	7.9
				2.0	1.55	0.51	0.25	11.0
				3.0	1.50	0.48	0.24	29.6
				4.0	1.45	0.48	0.24	29.6
				5.0	1.50	0.47	0.24	52.6
				6.0	1.40	0.48	0.22	64.3
				7.0	1.40	0.48	0.22	71.5
				8.0	1.40	0.48	0.22	86.4

* Change in the influent turbidity resulted from feed of a new sample of wastewater.

1* - Filter hydraulic loading = 1.4 $l/m^2.s$ (2.0 gpm/ft^2)

2* - Filter hydraulic loading = 4.1 $l/m^2.s$ (6.0 gpm/ft^2)

changed to 1.4 and 2.7 $l/m^2.s$ (2.0 and 6.0 gpm/ft^2). The effect on filtered effluent turbidity was not very great, but headloss increased and decreased according to increase and decrease in the filter hydraulic loading.

4.4 PERFORMANCE OF TREATMENT SYSTEM FOR CHLORINATED SECONDARY EFFLUENT

The same study was performed for chlorinated secondary effluent. Table 4.7 lists the data obtained for chlorinated secondary effluent at filter hydraulic loading of 2.7 $l/m^2.s$ (4.0 gpm/ft^2). Maximum influent turbidity was about 4.5 NTU and minimum filtered turbidity was 1.1 NTU. Better effluent turbidity was obtained at higher polymer dosage. High influent turbidity was due to the holding pond existing at the ARAMCO Wastewater Treatment Plant. Effluent is drawn from this pond for the final disposition after chlorination. Table 4.8 lists the data obtained for chlorinated secondary effluent when the filter hydraulic loadings were 1.4 and 2.7 $l/m^2.s$ (2.0 and 6.0 gpm/ft^2). High wastewater turbidity was due to color caused by a shock load on the treatment plant those days. Effluent settled and filtered turbidity was also high for the same reason. Filter was stopped before scheduled time due to high head loss at 4.1 $l/m^2.s$ (6.0 gpm/ft^2) filter hydraulic loading.

Table 4.7 : Experimental Data Obtained for Chlorinated Secondary Effluent [Filter Loading = 2.7 l/m².s (4.0 gpm/ft²)].

Test Ser.	No. of Runs	Chemical dosage (mg/l) Alum	Polymer	Time (hr)	Avg. Turbidity (NTU) Influ-ent	Settl-ed	Filter-ed	Avg. Head loss (cm)
1	3	5.0	0.10	0.0	3.70	1.70	0.82	0.0
				0.5	3.65	1.55	0.78	2.8
				1.0	3.55	1.50	0.74	4.8
				1.5	3.40	1.50	0.74	7.2
				2.0	3.40	1.50	0.75	8.2
				3.0	4.25*	1.45	0.77	10.1
				4.0	4.40	1.40	0.72	14.2
				5.0	4.10	1.40	0.73	17.9
				6.0	3.65	1.45	0.71	22.3
				7.0	3.45	1.40	0.71	27.6
				8.0	3.25	1.35	0.72	33.8
2	2	10.0	0.10	0.0	2.90	1.35	1.00	0.0
				0.5	2.80	1.25	0.89	3.6
				1.0	2.75	1.25	0.75	5.1
				1.5	2.70	1.25	0.77	7.0
				2.0	2.70	1.25	0.76	8.2
				3.0	3.40*	1.20	0.72	11.0
				4.0	3.25	1.20	0.70	13.6
				5.0	3.20	1.20	0.69	15.9
				6.0	3.45	1.25	0.68	18.4
				7.0	3.40	1.20	0.66	20.5
				8.0	3.35	1.20	0.65	24.5
3	3	20.0	0.1	0.0	3.40	1.55	0.86	0.0
				0.5	3.30	1.45	0.76	3.1
				1.0	3.20	1.45	0.70	4.0
				1.5	3.30	1.40	0.63	11.9
				2.0	3.30	1.45	0.65	6.9
				3.0	3.15	1.40	0.63	9.3
				4.0	3.10	1.35	0.63	11.9
				5.0	4.00*	1.35	0.61	14.5
				6.0	3.75	1.35	0.60	16.6
				7.0	3.65	1.35	0.61	29.6
				8.0	3.65	1.35	0.61	22.3

*Cont.

4	2	5.0	0.2	0.0	1.45	1.05	0.27	0.0
				0.5	1.45	1.02	0.19	1.5
				1.0	1.45	1.01	0.16	2.3
				1.5	1.30	1.00	0.16	3.7
				2.0	1.15	1.02	0.18	5.0
				3.0	1.15	1.02	0.15	7.4
				4.0	1.15	1.02	0.11	9.3
				5.0	1.15	0.97	0.10	11.0
				6.0	1.15	1.01	0.15	14.3
				7.0	1.15	1.05	0.16	16.8
				8.0	1.15	1.05	0.18	18.8
5	2	10.0	0.2	0.0	1.80	1.05	0.30	0.0
				0.5	1.70	1.05	0.23	2.3
				1.0	1.65	1.15	0.23	3.0
				1.5	1.75	1.05	0.17	3.7
				2.0	1.75	1.05	0.15	4.3
				3.0	1.75	1.10	0.13	5.3
				4.0	1.65	1.10	0.13	6.2
				5.0	1.60	1.05	0.12	7.7
				6.0	2.50*	1.00	0.13	9.2
				7.0	2.30	1.15	0.12	11.2
				8.0	2.25	1.15	0.13	13.2
6	2	10.0	0.3	0.0	2.30	0.85	0.27	0.0
				0.5	2.30	1.01	0.21	2.0
				1.0	2.35	1.08	0.28	2.4
				1.5	2.35	1.06	0.30	2.9
				2.0	2.25	1.06	0.30	3.5
				3.0	2.20	0.91	0.15	4.9
				4.0	2.15	0.86	0.12	6.2
				5.0	2.10	0.85	0.11	7.9
				6.0	2.40	0.85	0.11	9.8
				7.0	2.25	0.86	0.11	11.2
				8.0	2.10	0.86	0.11	12.3

* Change in the influent turbidity resulted from feed of a new sample of wastewater.

Table 4.8 : Experimental Data Obtained for Chlorinated Secondary Effluent with varying Filter Hydraulic Loadings

Test Ser.	No. of Runs	Chemical dosage (mg/l) Alum	Polymer	Time (hr)	Avg. Turbidity (NTU) Influent	Settled	Filtered	Avg. Head loss (cm)
1*	2	10.0	0.3	0.0	1.65	0.82	0.33	0.0
				0.5	1.65	0.81	0.16	0.9
				1.0	1.65	0.76	0.14	1.6
				1.5	1.65	0.84	0.18	2.3
				2.0	1.65	0.83	0.16	3.1
				3.0	4.10*	0.82	0.14	3.9
				4.0	4.10	0.91	0.14	4.8
				5.0	4.10	0.80	0.15	5.7
				6.0	4.00	0.77	0.17	7.3
				7.0	3.90	0.76	0.17	11.3
				8.0	3.90	0.76	0.16	13.4
2*	2	10.0	0.3	0.0	4.80	1.00	0.62	0.0
				0.5	4.80	1.00	0.60	6.5
				1.0	4.80	1.00	0.60	11.5
				1.5	4.80	0.99	0.60	18.8
				2.0	4.65	1.15	0.60	37.0
				3.0	6.00	1.30	0.98	71.6
				4.0	5.65	1.35	0.97	110.9
				5.0	5.10	1.20	0.97	142.4
				6.0	5.20	1.20	0.97	164.5**

* New sample of wastewater had some colour.

** Filter was stopped before scheduled time because of very high headloss.

1* - Filter hydraulic loading = 1.4 $l/m^2.s$ (2.0 gpm/ft^2)

2* - Filter hydraulic loading = 4.1 $l/m^2.s$ (6.0 gpm/ft^2)

4.5 RESULTS OF BATCH CHLORINATION AND COLIFORM COUNT

Batch chlorination was performed on filtered secondary effluent only. Table 4.9 gives the chlorine residual and the coliform count at contact times 10, 20, 40 and 80 minute for a chlorine dosage of 10 mg/l. This concentration was chosen based on the chlorine dosage used in the Pomona Virus Study (12). The maximum number of coliforms found in one test was 10, while in most of the cases the coliform count was less than 2 per 100 ml.

The majority of the coliforms were removed by the filtration process. Average coliform count in the raw wastewater was approximately 1.5×10^5 per 100 ml and in the filtered effluent it was about 3000 coliforms per 100 ml.

Table 4.9 : Summary of Calculated and Measured Data for chlorination Process and Total Coliform Count

Average coliform concentration after filtration : 3000 per 100 ml.

Test Series No.	Applied Chlorine Dosage (mg/l)	Contact Time(CT) (min)	Residual Chlorine(R) (mg/l)	CT * R (min-mg/l)	Total Coliforms MPN/100 ml
1	10	10	6.0	60	10
		20	5.2	104	5
		40	4.4	176	4
		80	3.2	256	3
2	10	10	6.6	66	4
		20	6.0	120	2
		40	5.4	120	2
		80	4.8	384	0
3	10	10	8.0	80	1
		20	-	-	-
		40	6.8	272	3
		80	6.6	528	1
4	10	10	7.4	74	2
		20	6.8	136	1
		40	6.4	256	-
		80	6.0	480	-

*Cont.

5	10	10	7.6	76	3
		20	7.0	140	1
		40	6.3	252	1
		80	5.3	424	1
6	10	10	7.8	78	3
		20	7.2	144	1
		40	6.2	248	2
		80	5.2	416	-

Chapter 5

DISCUSSION

This chapter discusses the specific topics including : turbidity removal by sedimentation and filtration; effect of filtration rate on filter performance; disinfection of the filtered effluent; and overall system performance. Table 5.1 summarizes the experimental data obtained for unchlorinated and chlorinated secondary effluent. The table gives the chemical costs; average influent, settled and filtered turbidities; overall turbidity removal by sedimentation and filtration; and maximum headloss developed in 8 hr of filter run at various chemical dosages for both kinds of wastewater. These data are given here for reference in subsequent discussions.

5.1 TURBIDITY REMOVAL BY SEDIMENTATION AND FILTRATION

This section deals with the discussion of turbidity removal by sedimentation and filtration for the two kinds of wastewater separately.

5.1.1 Unchlorinated Secondary Effluent

The overall percentage of turbidity removal for unchlorinated wastewater ranged from 25 to 61 after sedimentation while it ranged from 72 to 80 % after the filtration process, at various chemical dosages as listed in Table 5.1. The graphs of influent-settled and influent-filtered turbidities versus filtration time are illustrated in Fig. 5.1 and Fig. 5.2 respectively (See Table 4.5 for the data). Influent turbidity

Table 5.1 : Summary of the Experimental Data Obtained for Unchlorinated and Chlorinated Secondary Effluent

Chemical Dosage (mg/l)		Cost of Chemicals	Average Turbidity (NTU)			Overall Turbidity Removal (%)		Turbidity Removal by Filter Alone (%)	Headloss in 8 hr (cm)
Alum	Polymer	SR/1000m^3	Influent	Settled	Filtered	After Sedimentation	After Filtration		
Unchlorinated Secondary Effluent									
5.0	0.1	17.2	1.10	0.70	0.31	36	72	36	30.1
10.0	0.1	32.2	1.10	0.83	0.23	25	79	54	20.4
5.0	0.2	19.4	1.15	0.68	0.30	41	74	33	38.5
10.0	0.2	34.4	1.25	0.62	0.26	50	80	30	38.3
5.0	0.3	21.6	1.65	0.67	0.36	59	78	19	27.6
10.0	0.3	36.6	1.14	0.45	0.25	61	78	17	107.8
Chlorinated Secondary Effluent									
5.0	0.1	17.2	3.76	1.46	0.74	61	80	19	33.8
10.0	0.1	32.2	3.14	1.23	0.73	61	77	16	24.5
20.0	0.1	62.2	3.46	1.39	0.65	60	81	21	22.3
5.0	0.2	19.4	1.21	1.02	0.16	16	87	71	18.8
10.0	0.2	34.4	1.83	1.08	0.15	41	92	51	13.2
10.0	0.3	36.6	2.23	0.92	0.17	59	92	33	12.3

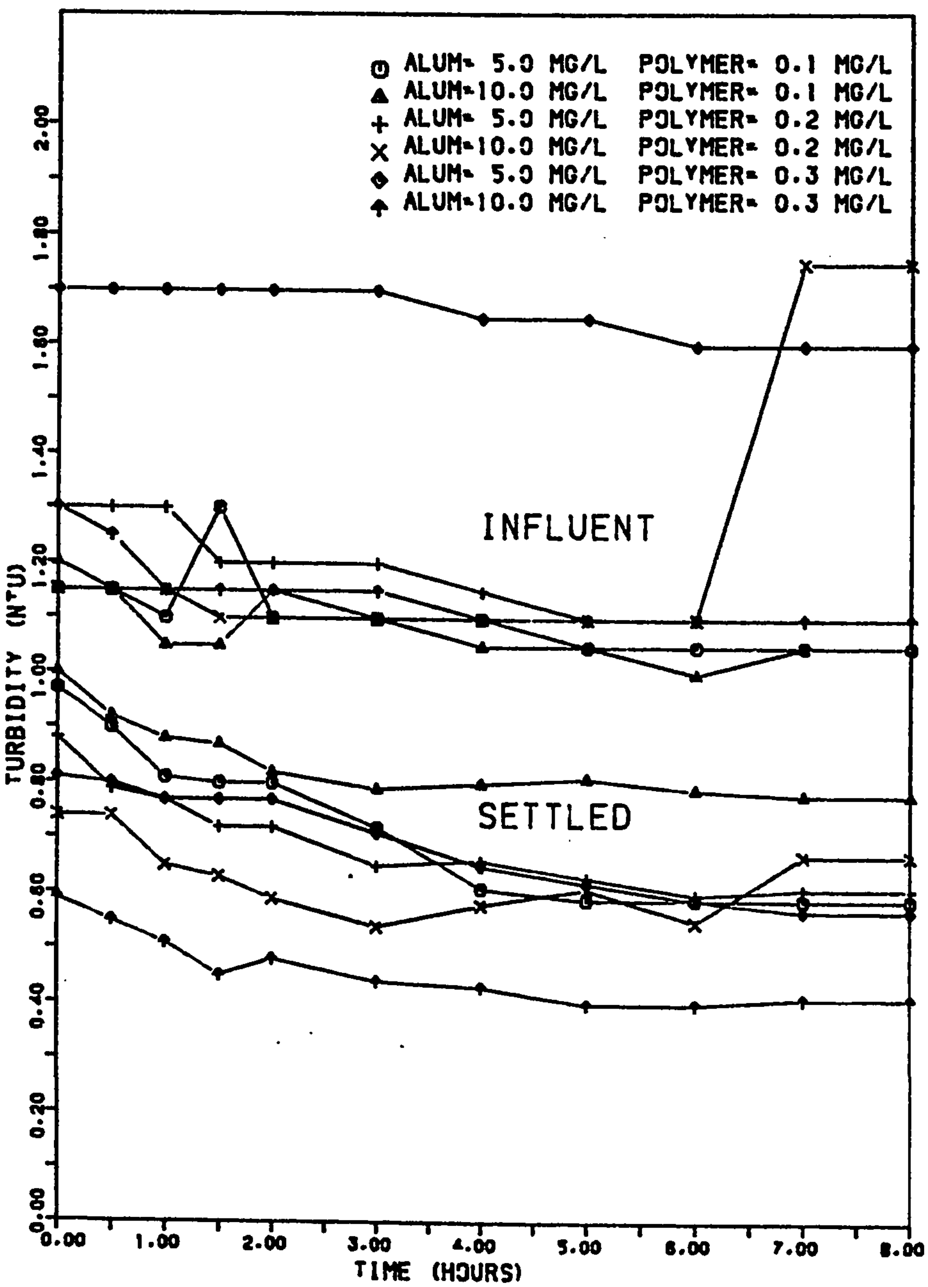

Fig. 5.1 : Influent and Settled Turbidities Versus Time at Various Chemical Dosages for Unchlorinated Secondary Effluent

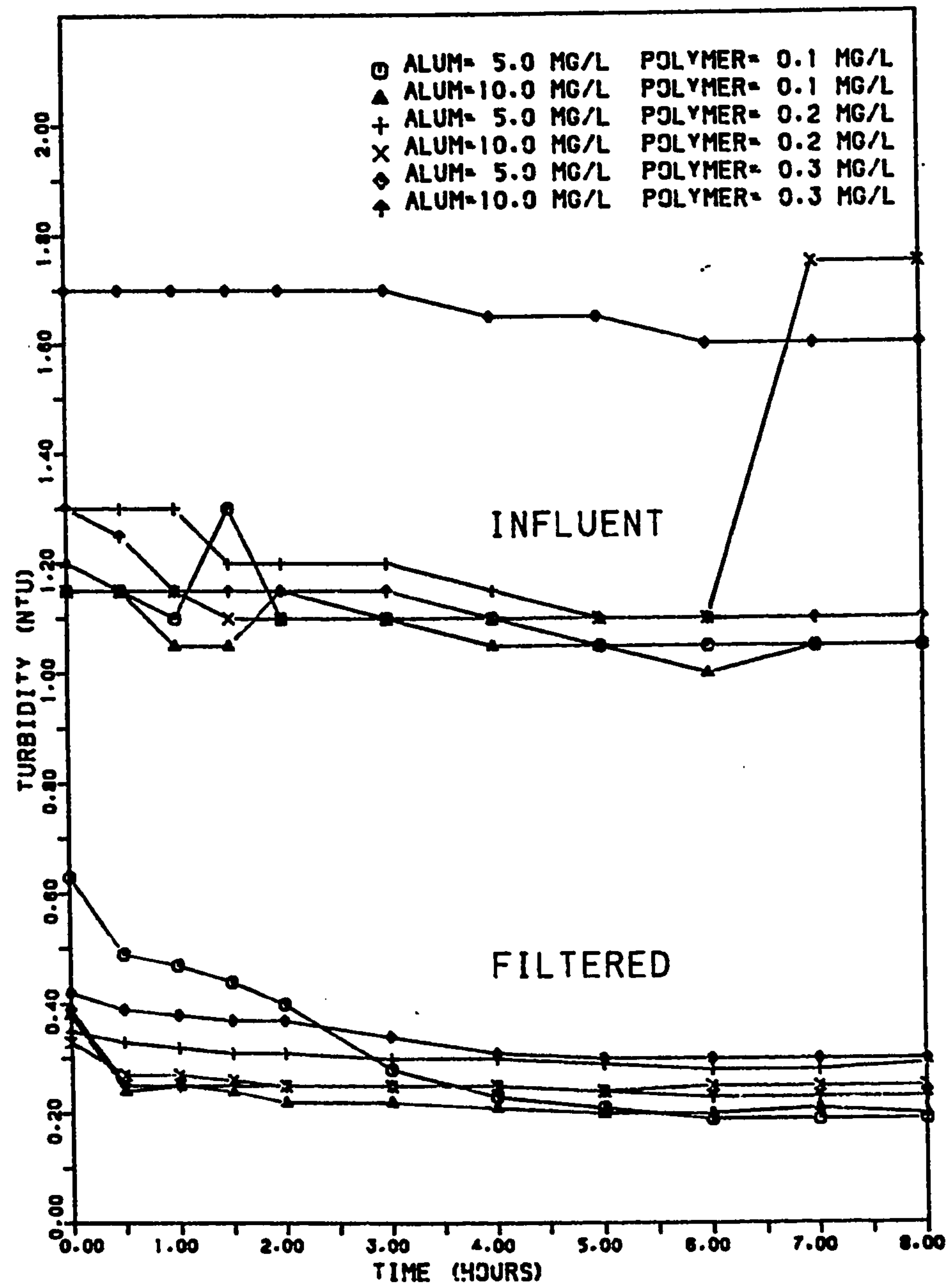

Fig. 5.2 : Influent and Filtered Turbidities Versus Time at Various Chemical Dosages for Unchlorinated Secondary Effluent at a Filtration Rate of 2.7 $1/m^2.S$ (4.0 gpm/ft^2)

was between 1.0 and 2.0 NTU all the time. The average minimum settled and filtered turbidities were obtained as 0.45 NTU and 0.25 NTU, respectively. Fig. 5.3 illustrates the headloss versus time at different chemical dosages. The headloss obtained was between 20 to 40 cm except in one case when it went very high. The reason was the excess polymer that clogged the filter media. Fig. 5.4 shows the overall percentage of turbidity removal versus chemical cost in Saudi Riyals per thousand cubic meters. The cost ranges from 17 to 37 Saudi Riyals. As illustrated in Fig. 5.4, after sedimentation 50 to 60 percent of the turbidity removal was obtained with three chemical dosages that are alum and polymer as : 5 mg/l, 0.3 mg/l; 10 mg/l, 0.2 mg/l; and 10 mg/l, 0.3 mg/l. The maximum turbidity removal after sedimentation was at the chemical dosage of 10 mg/l alum and 0.3 mg/l polymer, but the overall turbidity removal after filtration was higher in case of 10 mg/l alum and 0.2 mg/l polymer. These two chemical dosages gave almost the same effluent quality in both bench scale study and jar testing. However the headloss obtained at the chemical dosage of 10 mg/l alum and 0.3 mg/l polymer was three times the headloss at the chemical dosage 10 mg/l alum and 0.2 mg/l polymer (see Fig. 5.5 and Table 5.1). This high headloss was due to unused polymer carried over through the sedimentation tank and clogging the filter media. Satisfactory turbidity removal after sedimentation and filtration and minimum headloss was obtained at the chemical dosage of 5 mg/l alum and 0.3 mg/l polymer. The large floc formed at this chemical dosage, however, were very fragile and easily broken into smaller particles if the flocculator paddle

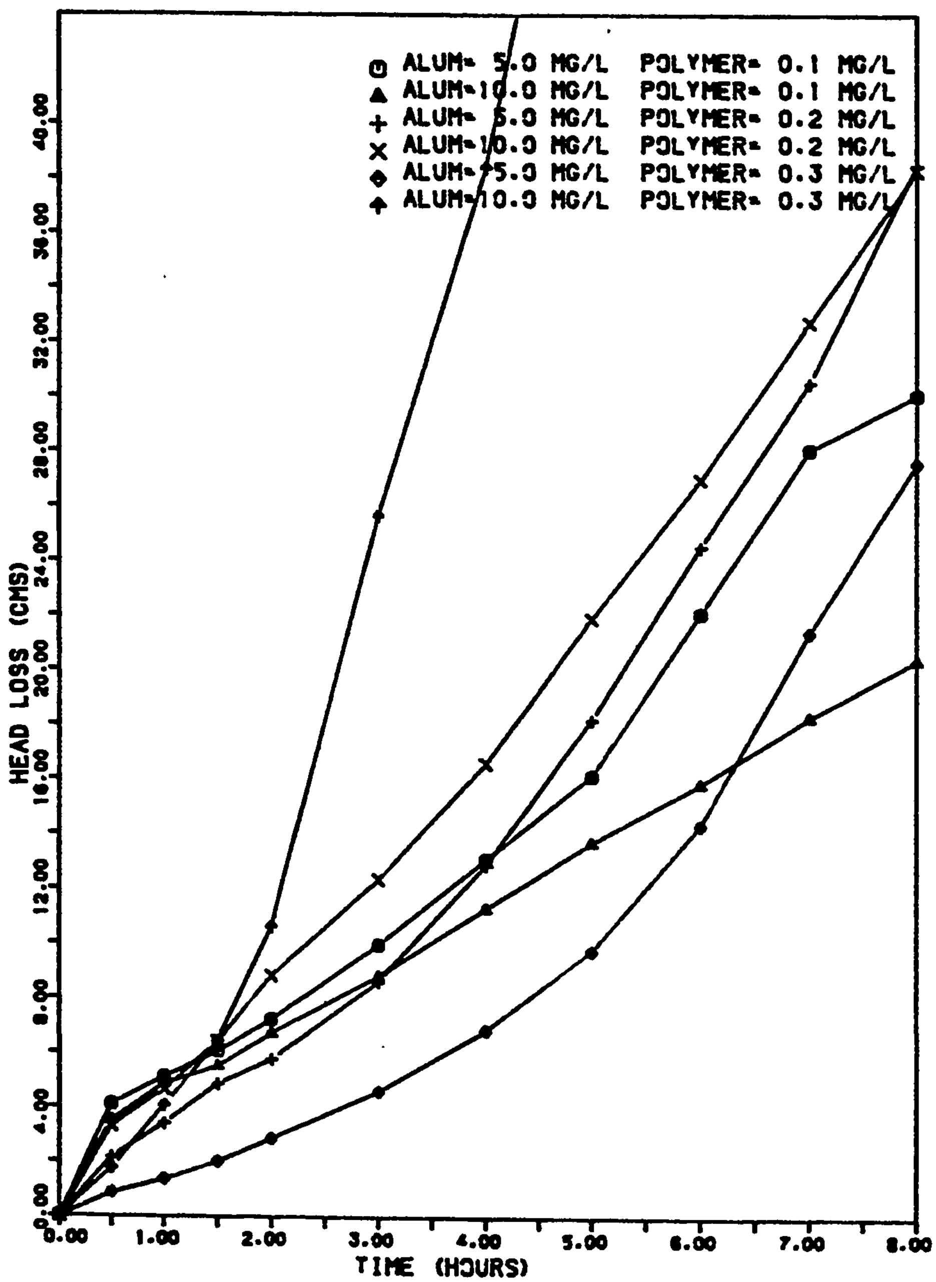

Fig. 5.3: Headloss Versus Time for Unchlorinated Secondary Effluent at Various Chemical Dosages

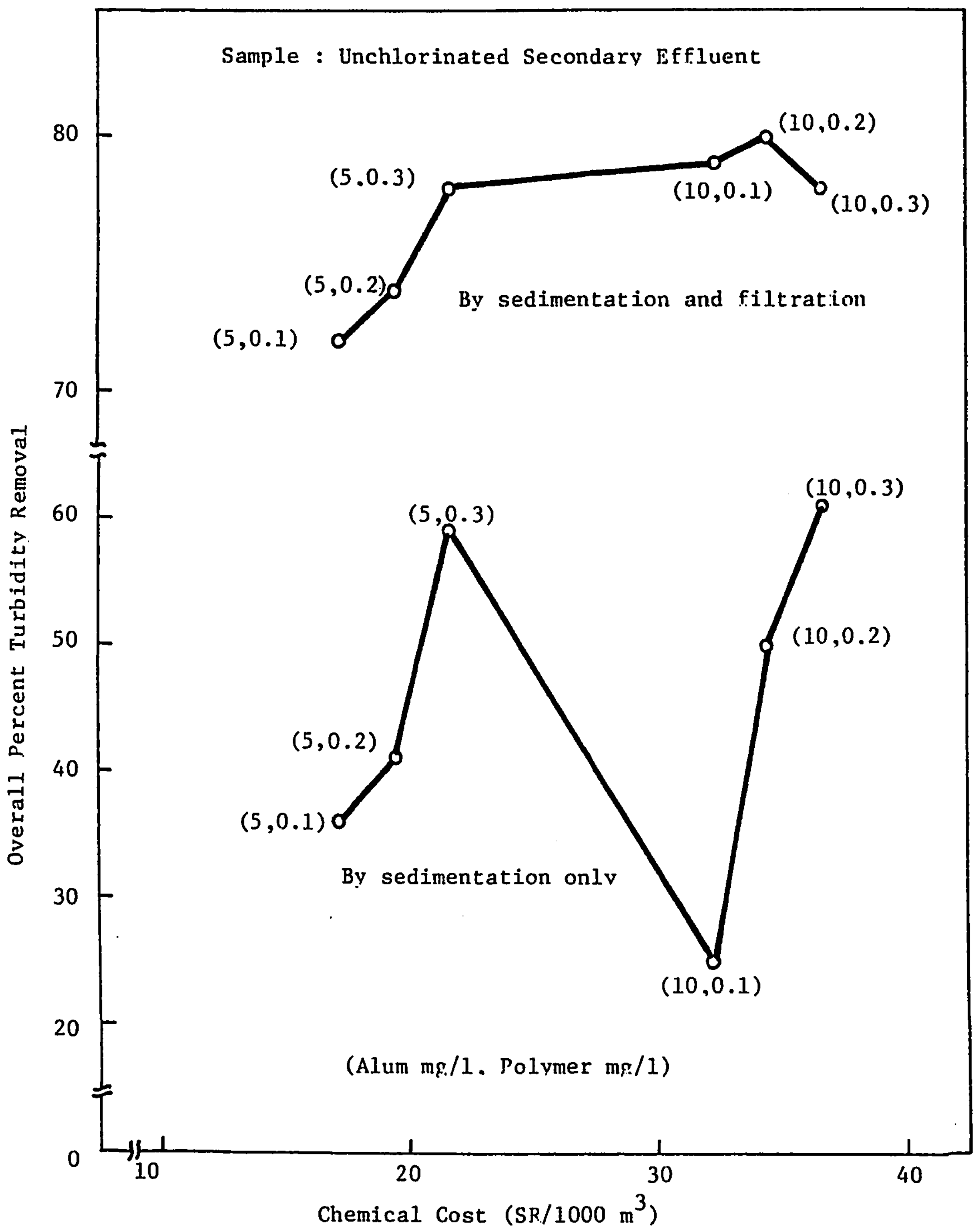

Fig. 5.4 : Turbidity Removal Versus Chemical Cost for Various Dosages

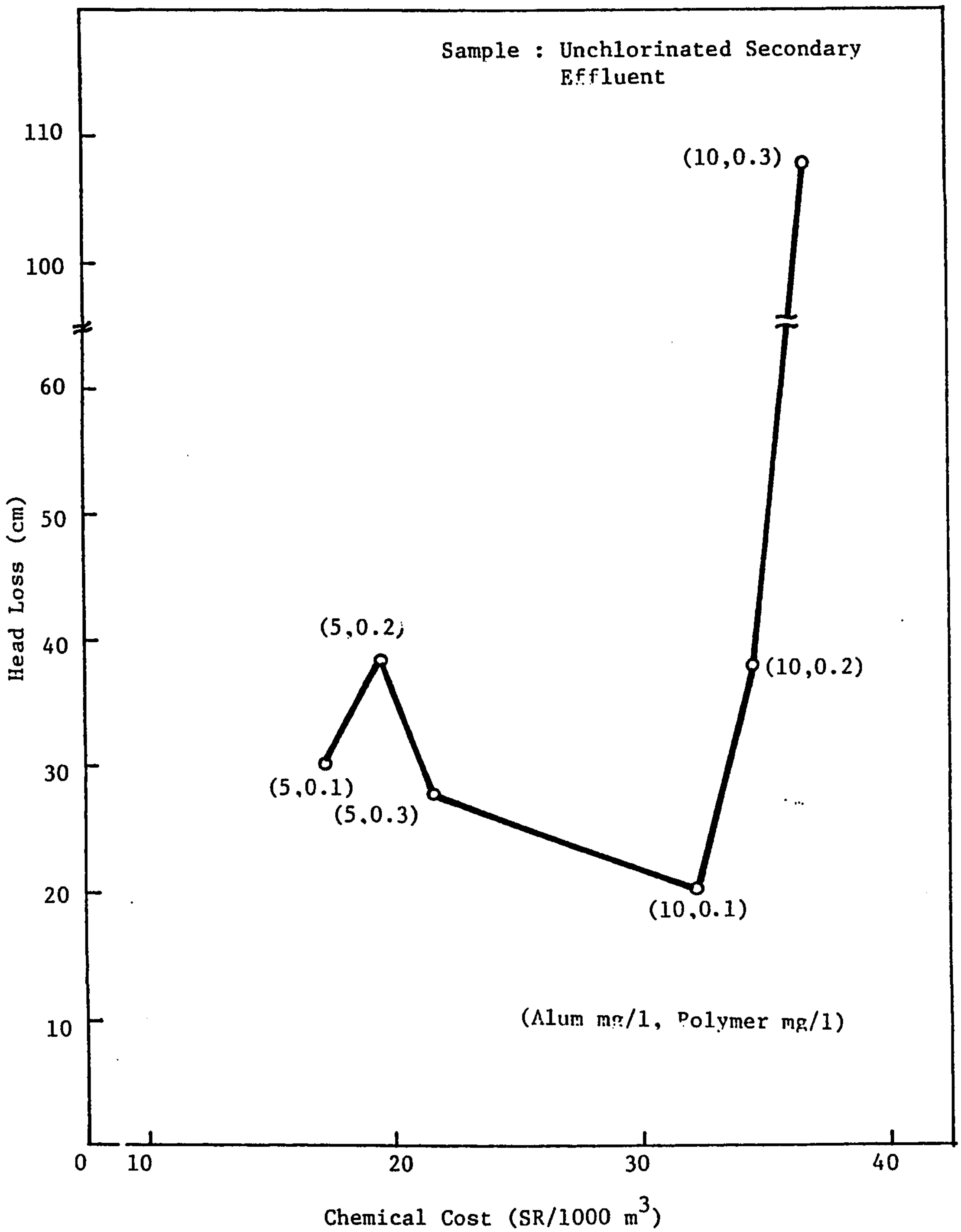

Fig. 5.5 : Head Loss Versus Chemical Cost of Various Dosages

speed was not carefully controlled. At the dosage of 10 mg/l alum and 0.2 mg/l polymer, the floc were stronger and less friable providing more reliable gravity separation of influent turbidity. Therefore, the optimum chemical dosage for sedimentation also depended on the alum to polymer ratio. The dosage of 10 mg/l alum and 0.2 mg/l polymer appears to be more suitable for coagulation prior to sedimentation to remove contaminants, such as, warm ova and protozoal cysts. Yet, from the view of overall removal in sedimentation and filtration, an alum dosage of 5 mg/l with 0.1 to 0.3 mg/l polymer produced an effluent of low turbidity at reduced chemical cost.

5.1.2 Chlorinated Secondary Effluent

This wastewater had a higher turbidity than the unchlorinated secondary effluent. The reason is that the effluent holding pond allowed growth of algae increasing the turbidity of wastewater. The results of treatment of this wastewater indicate that filtered effluent turbidity was satisfactory when the polymer dosage was 0.2 mg/l or higher with an alum dosage of either 5 or 10 mg/l (See Table 5.1). The graphs of influent-settled and influent-filtered turbidities versus time are illustrated in Fig. 5.6 and Fig. 5.7 respectively using the data tabulated in Table 4.7. The influent turbidity ranged from 1.0 NTU to 4.0 NTU while the average minimum settled and filtered effluent turbidities obtained were 0.92 NTU and 0.15 NTU respectively. Fig. 5.8 illustrates the headloss versus time at various chemical dosages. The headloss ranged between 10 to 40 cm. Fig. 5.9 illustrates the overall

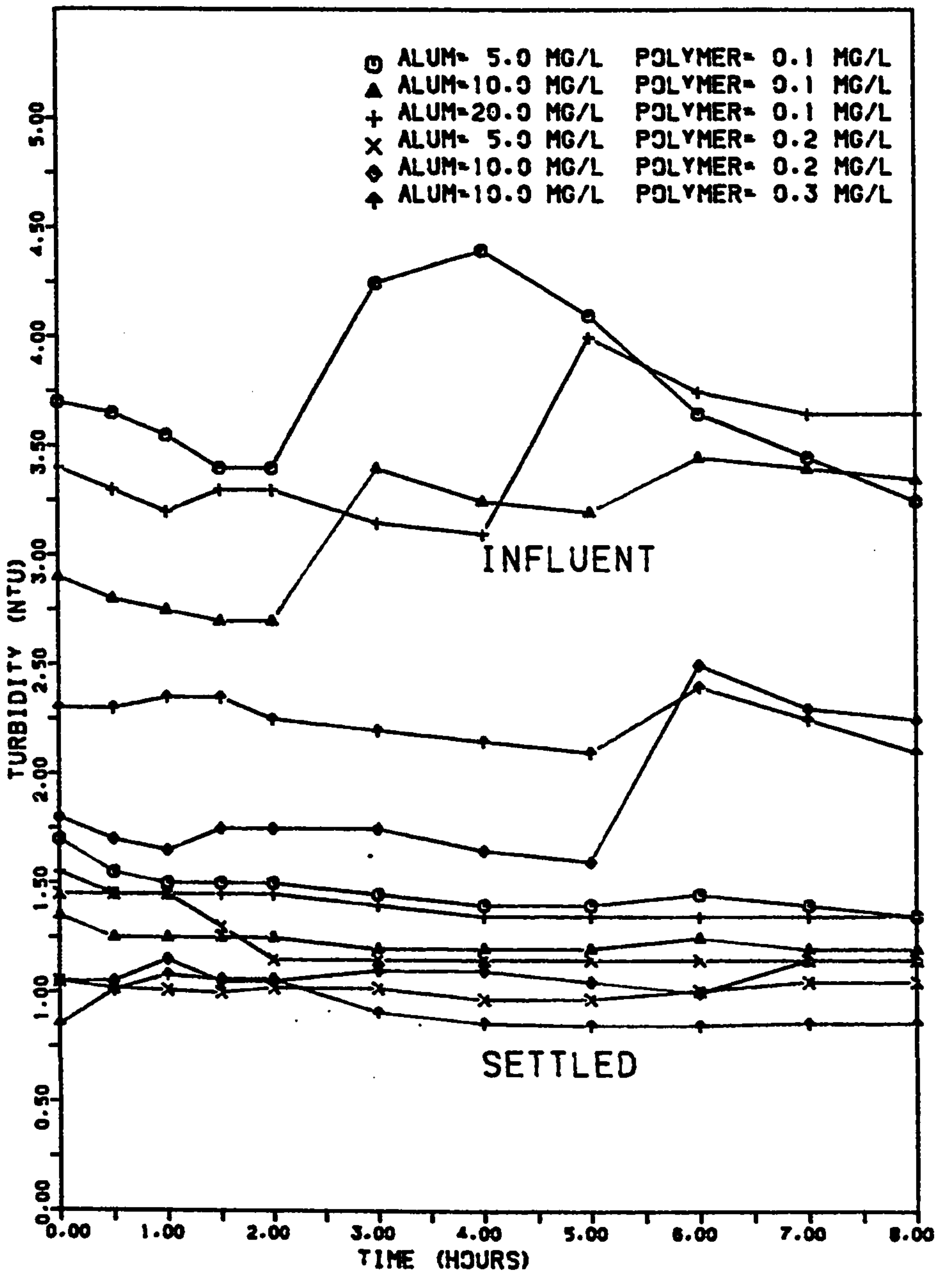

Fig. 5.6 : Influent and Settled Turbidities Versus Time at Various Chemical Dosage for Chlorinated Secondary Effluent

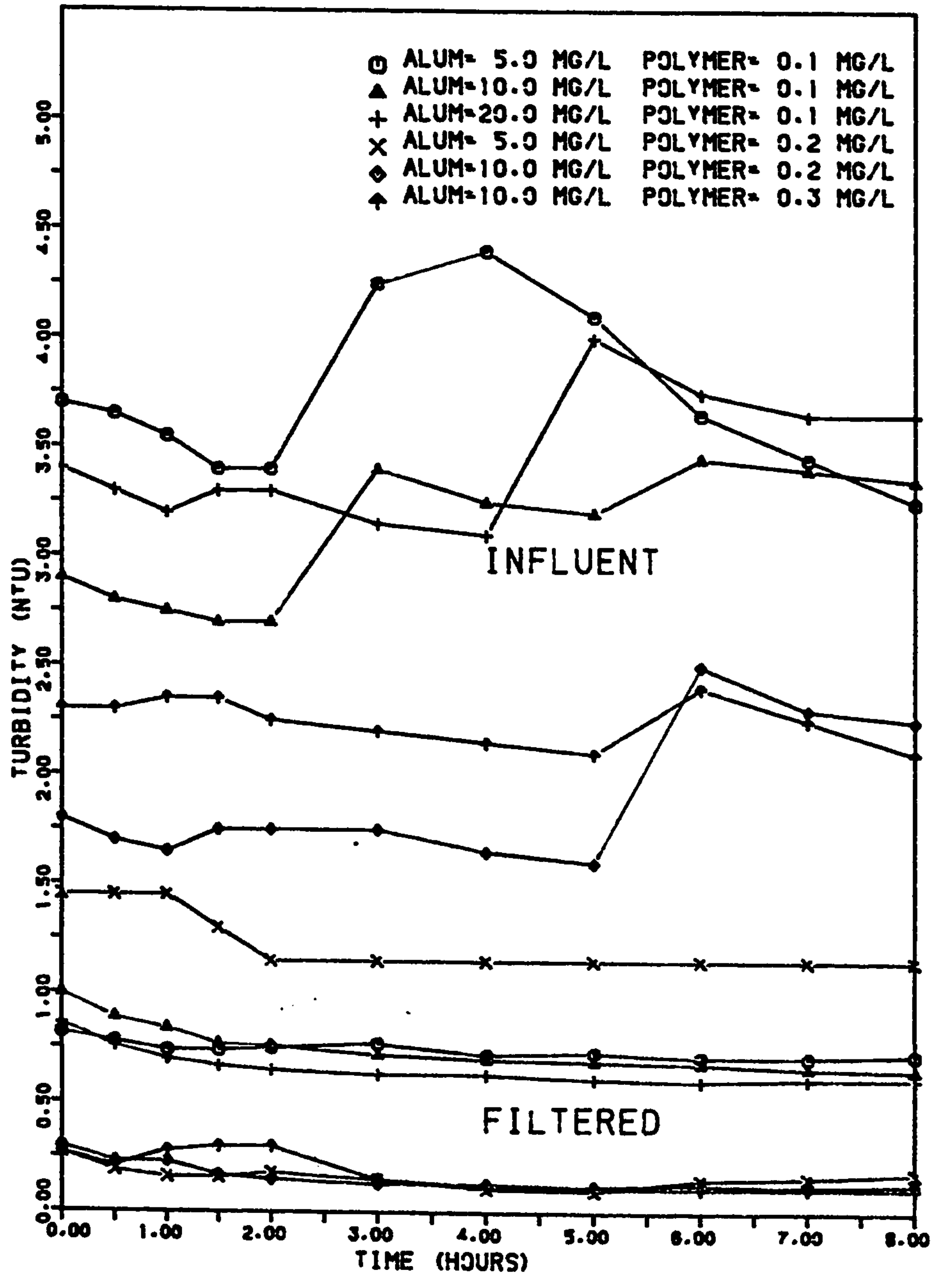

Fig. 5.7 : Influent and Filtered Turbidities Versus Time at Various Chemical Dosage for Chlorinated Secondary Effluent at 2.7 $l/m^2.S$ (4.0 gpm/ft^2)

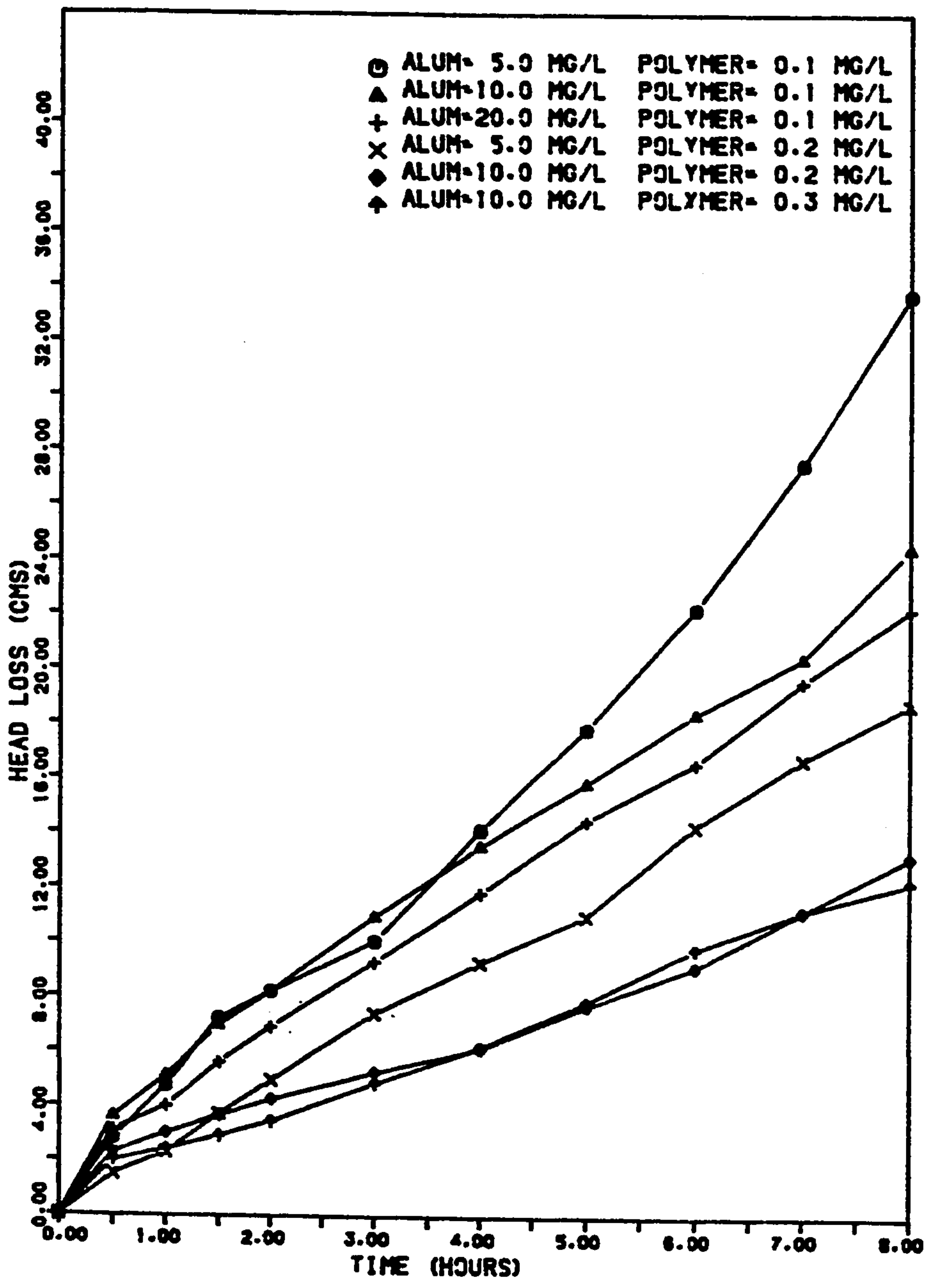

Fig. 5.8 : Headloss Versus Time at Various Chemical Dosages for Chlorinated Secondary Effluent

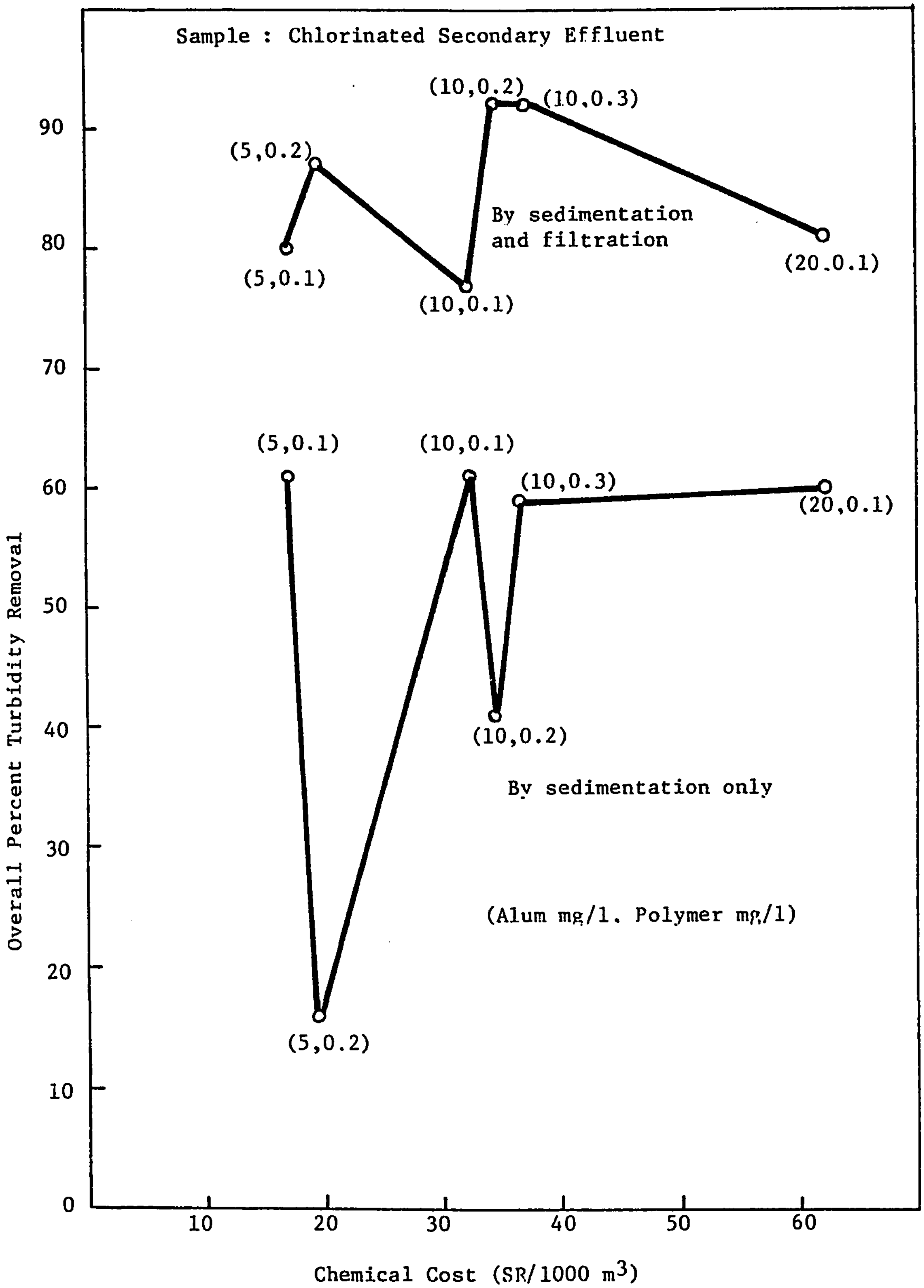

Fig. 5.9 : Turbidity Removal Versus Chemical Cost for Various Dosages

percentage of turbidity removal versus chemical cost. About 60 percent turbidity removal after sedimentation was obtained with four chemical dosages but the maximum overall turbidity removal after filtration occurred at the only two chemical dosages of 10 mg/l alum, 0.2 mg/l polymer and 10 mg/l alum, 0.3 mg/l polymer. The chemical costs for these two chemical dosages are almost similar. Comparing the sedimentation tank performance for these two chemical dosages, 10 mg/l alum and 0.3 mg/l polymer gave more satisfactory results than the chemical dosage of 10 mg/l alum and 0.2 mg/l polymer. Moreover the maximum headloss obtained in 8 hr of filter run was minimum for the chemical dosage 10 mg/l alum and 0.3 mg/l polymer (See Fig. 5.10). Based on the above discussion, the conclusion is that the optimum chemical dosage for chlorinated secondary effluent is 10 mg/l alum and 0.3 mg/l polymer. However a chemical dosage of 10 mg/l alum and 0.2 mg/l polymer also gave satisfactory results.

5.2 EFFECT OF FILTRATION RATE ON FILTER PERFORMANCE

To observe the effect of filtration rates on the turbidity removal and headloss development, the system was operated at different filter hydraulic loadings at the selected chemical dosages for unchlorinated and chlorinated secondary effluents. Fig. 5.11 and 5.12 illustrate the system performance for the chemical dosage of 10 mg/l alum and 0.2 mg/l polymer and at the hydraulic loadings of 1.4 $l/m^2.s$ (2.0 gpm/ft^2) and 4.1 $l/m^2.s$ (6.0 gpm/ft^2) respectively. Comparing these two graphs, the effluent quality was not affected by the flow rate upto 4.1

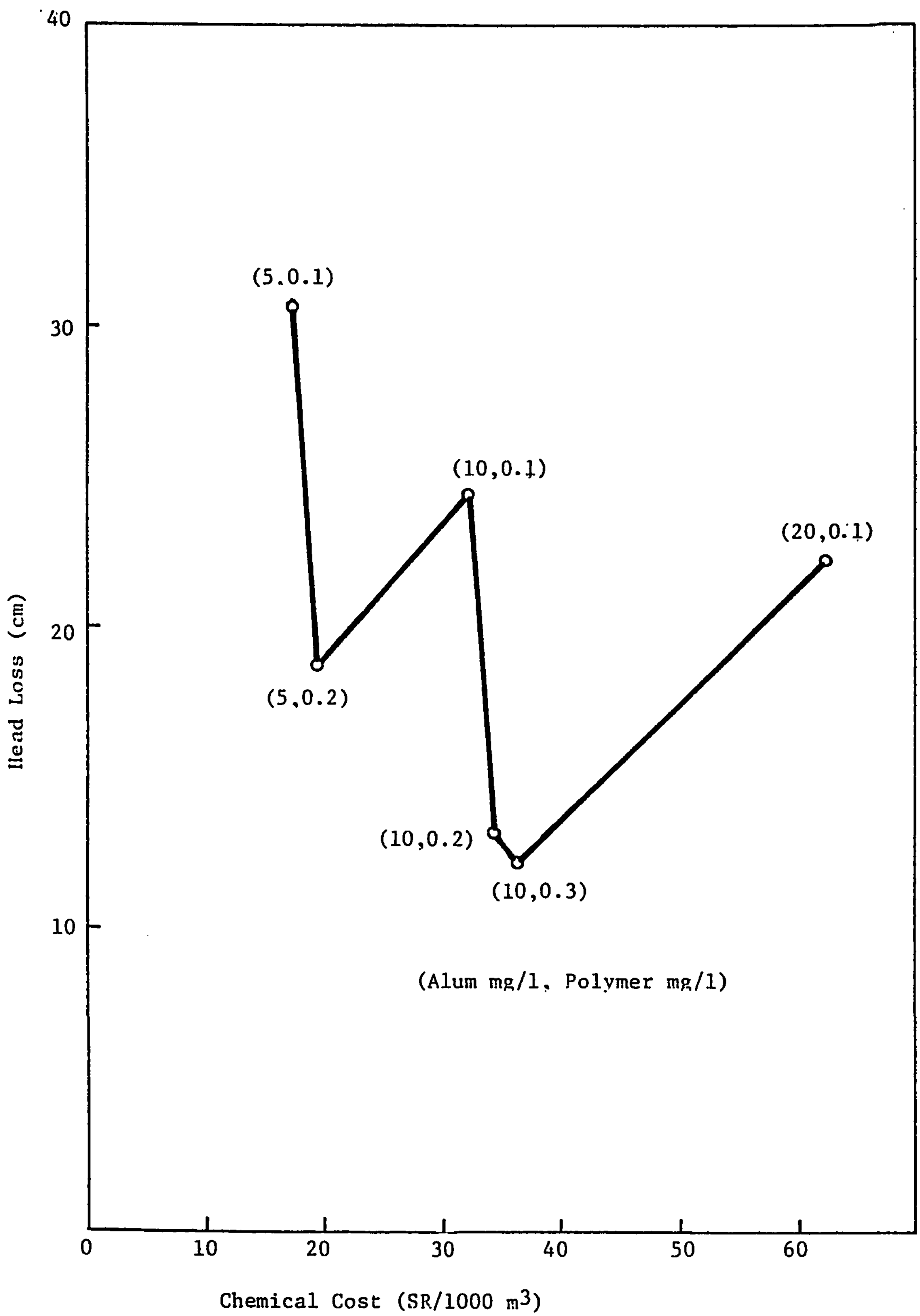

Fig. 5.10: Head Loss Versus Chemical Cost for Various Dosages

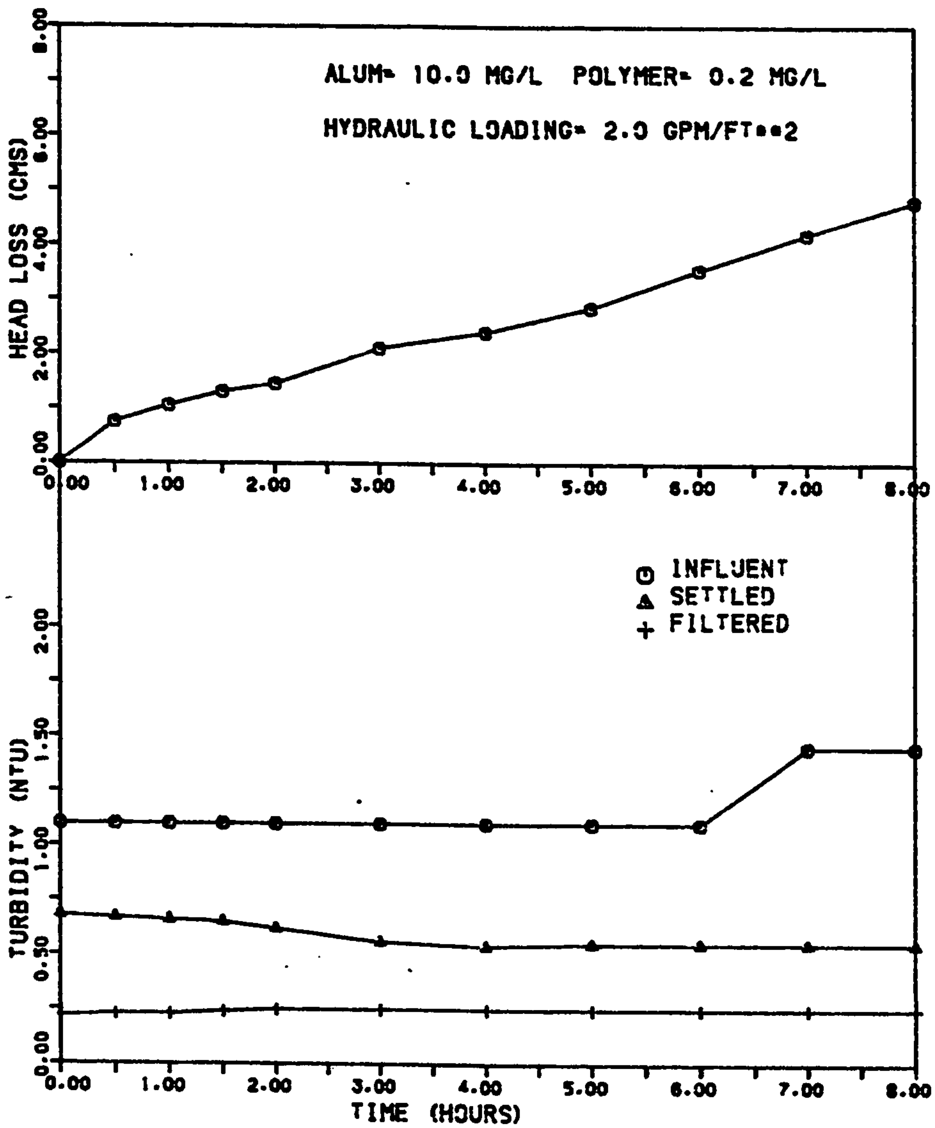

Fig. 5.11: System Performance at the Optimum Chemical Dosage for Unchlorinated Secondary Effluent at Hydraulic Loading 1.4 $l/m^2.S$ (2.0 gpm/ft^2)

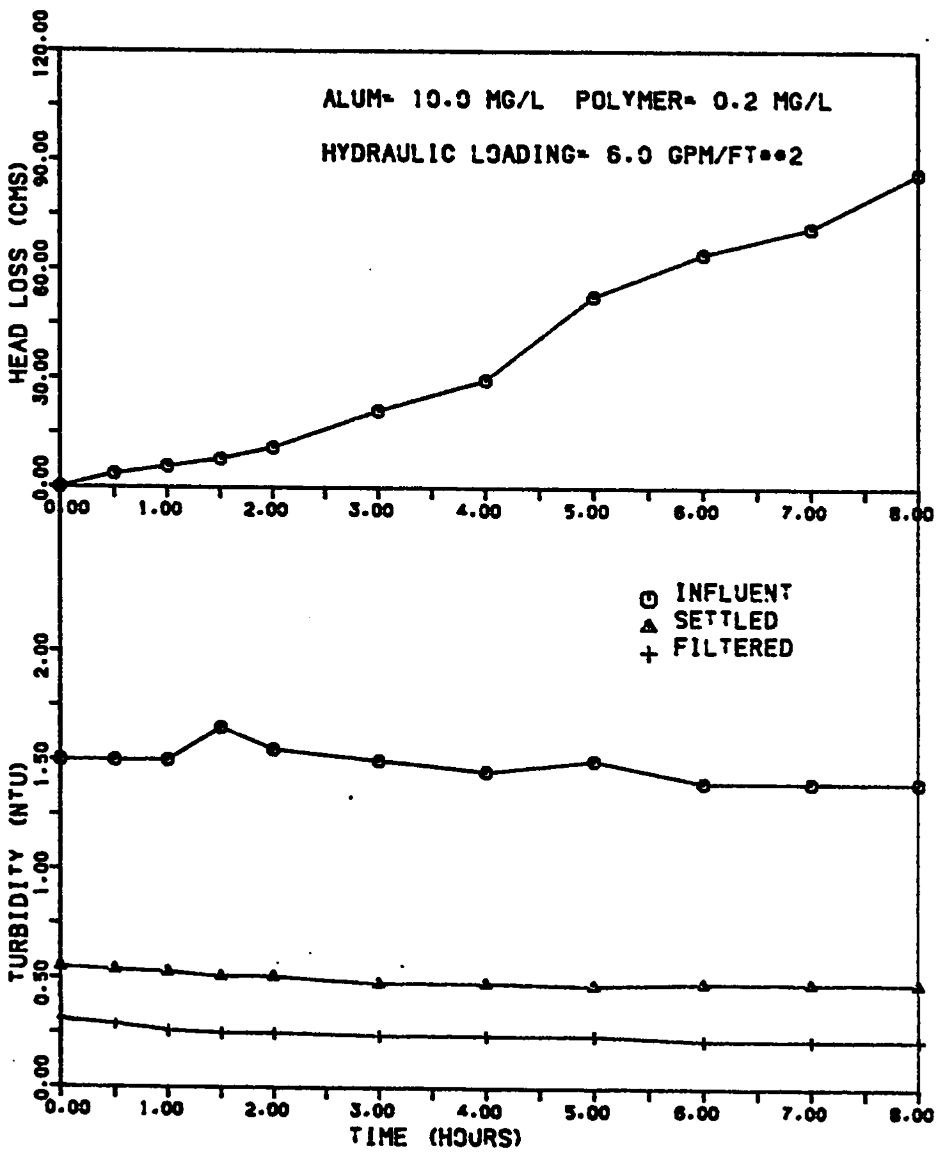

Fig 5.12 : System Performance at the Optimum Chemical Dosage for Unchlorinated Secondary Effluent at Hydraulic Loading 4.1 $l/m^2.S$ (6 gpm/ft^2)

l/m^2.s but the headloss was very high with the higher flow rate. The same conclusion can be drawn for the chlorinated secondary effluent comparing the Fig. 5.13 and Fig. 5.14. These results confirm the observations by Baumann and Huang (27).

The observation that a filtration rate upto 4.1 l/m^2.s does not lead to deterioration of the effluent quality means that more economical filter construction is possible. For example, if the flow rate is normally doubled, the area of the filters can be approximately halved resulting significant savings in the first cost of the required filter. Although effluent quality appears to be satisfactory at high filtration rates, according to recent studies a large number of submicroscopic particulates can be present in water of low turbidity. Therefore, the enthusiasm to push filtration rates higher than current practice should be tempered by the desire to produce high-quality water at all times (23).

A most useful relationship between net water production and run lengths obtained at different filtration rates is shown in Fig. 5.15 (33). Water production per run is computed by multiplying the filtration rates by different run lengths in hours assumed as 1, 2, 3, 5, 10, 20, 30, 50 and infinite duration. The assumed period for backwash is 30 minutes, including 3 minutes of air scour followed by a water wash of 5 minutes at 13.6 l/m^2.s (20 gpm/ft^2). Fig. 5.15 shows that there exists an upper limit of net water production at each filtration rate. The maximum net water production that can be obtained in a day is 117, 234, and 352 m^3/m^2 (2880, 5760 and 8640 gal/ft^2) for filtration rates of 1.4, 2.7 and 4.1 l/m^2.s (2.0, 4.0, and 6.0 gpm/ft^2) respectively.

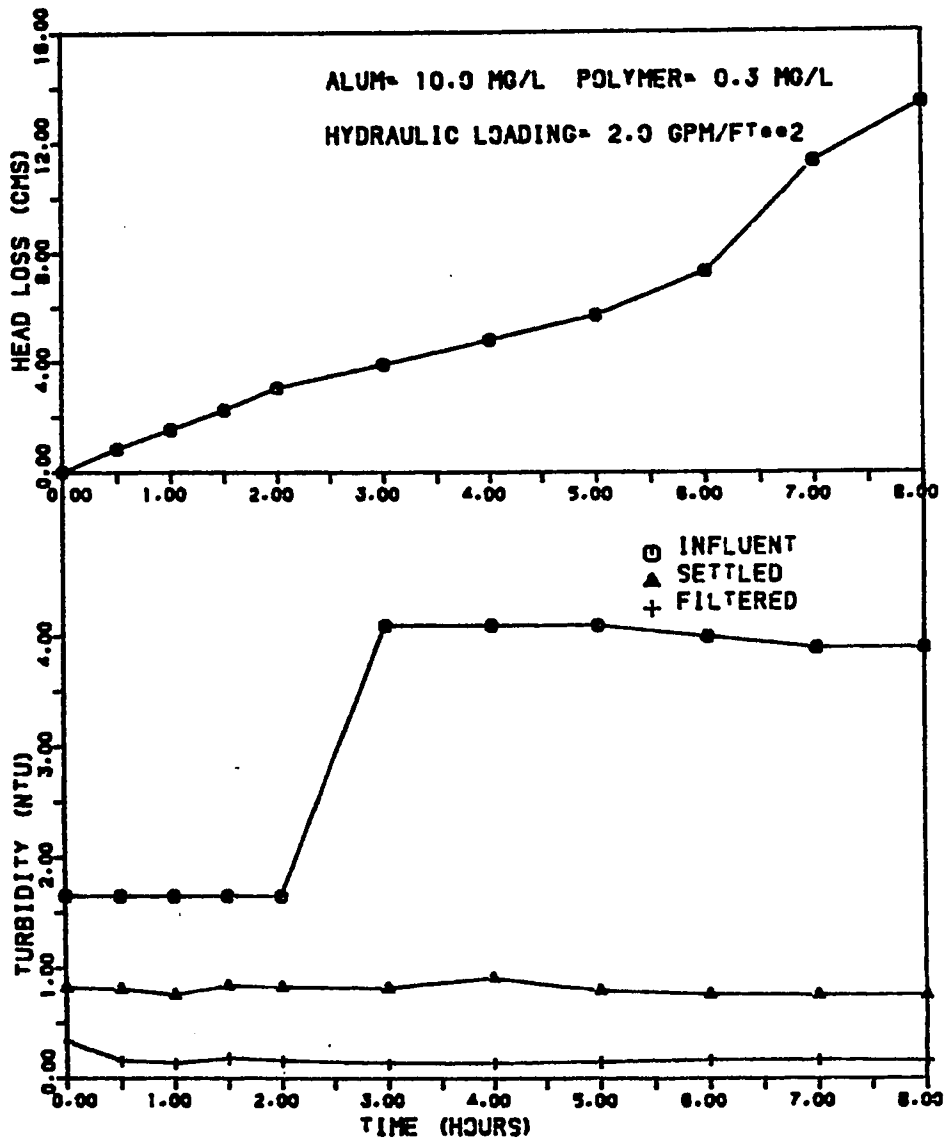

Fig. 5.13 : System Performance at the Optimum Chemical Dosage for Chlorinated Secondary Effluent at Hydraulic Loading 1.4 $l/m^2.S$ (2.0 gpm/ft^2)

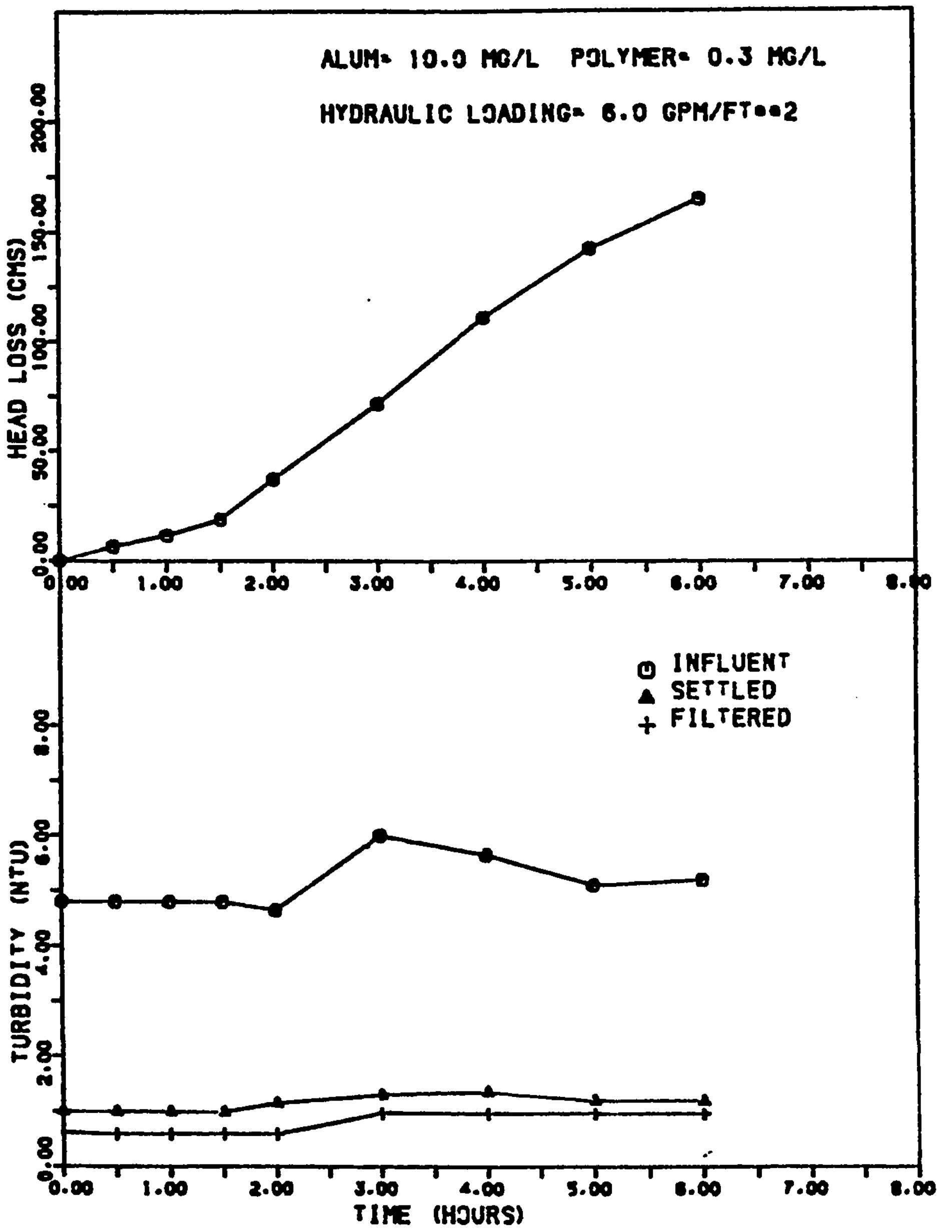

Fig. 5.14 : System Performance at the Optimum Chemical Dosage for Chlorinated Secondary Effluent at Hydraulic Loading 4.1 $l/m^2.S$ (6.0 gpm/ft^2)

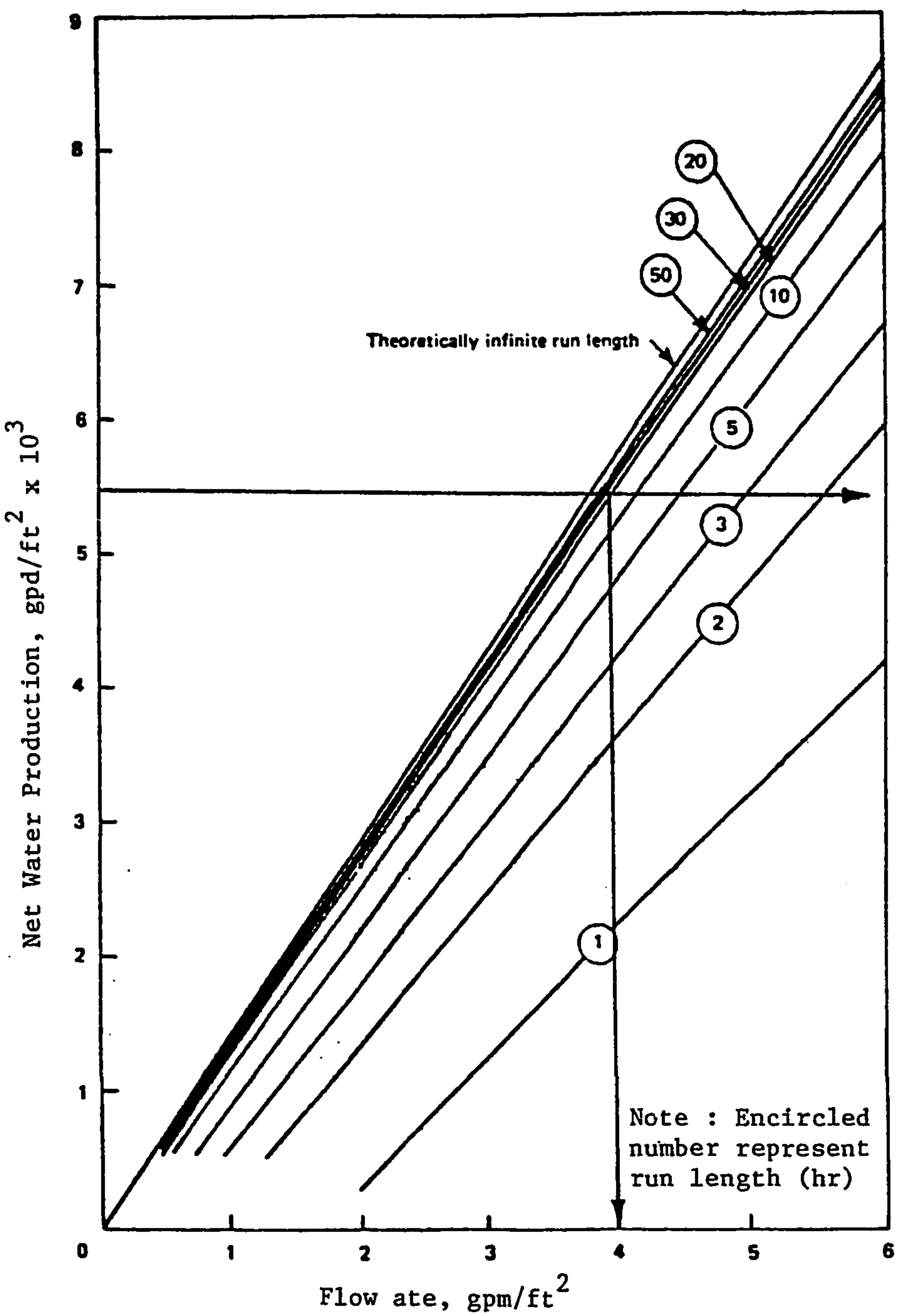

Fig. 5.15 : Net Water Production Versus Flow Rate at Various Run Lengths Assuming 30 Minute Backwashing Period (Adopted from 33)

Assume a net water production of 224 $m^3/d/m^2$ (5500 gpd/ft^2) is required. This is unattainable with any filtration rate less than 2.5 $l/m^2.s$ (3.75 gpm/ft^2), even with an infinite run length. At 3.1 $l/m^2.s$ (4.5 gpm/ft^2) a run length of 5 hr is needed. At 3.4 $l/m^2.s$ (5 gpm/ft^2) a run length of only 3 hr is needed. This amount can also be attained in 24 hr at 2.7 $l/m^2.s$ (4.0 gpm/ft^2).

North ARAMCO Wastewater Treatment Plant is designed to treat 8 million gallons of wastewater per day (30280 m^3/d). If four filters of the size 8 m x 4.5 m (Area = 144 m^2) are provided then the total net production of water at 2.7 $l/m^2.s$ (4.0 gpm/ft^2) will be 32200 m^3/d that will take care of the plant capacity. Therefore the recommended flow rate is 2.7 $l/m^2.s$ (4.0 gpm/ft^2).

5.3 DISINFECTION OF EFFLUENT

Portions of unchlorinated filtered effluent were batch chlorinated to predict an experimentally derived model that relates coliform inactivation to the product of contact time (CT) and chlorine residual (R). Table 4.9 summarizes the calculated and measured data for the chlorination process and total coliform count. A maximum 10 coliforms were detected in one test. Fig. 5.16 illustrates the chlorination model for this study. The highest points are taken for fitting the curve because of the safety factor. The product of contact time and residual chlorine is approximately 430 to achieve an effluent coliform concentration of less than 2.2 MPN/100 ml.

The contact time-residual chlorine value was found to be 1000 in

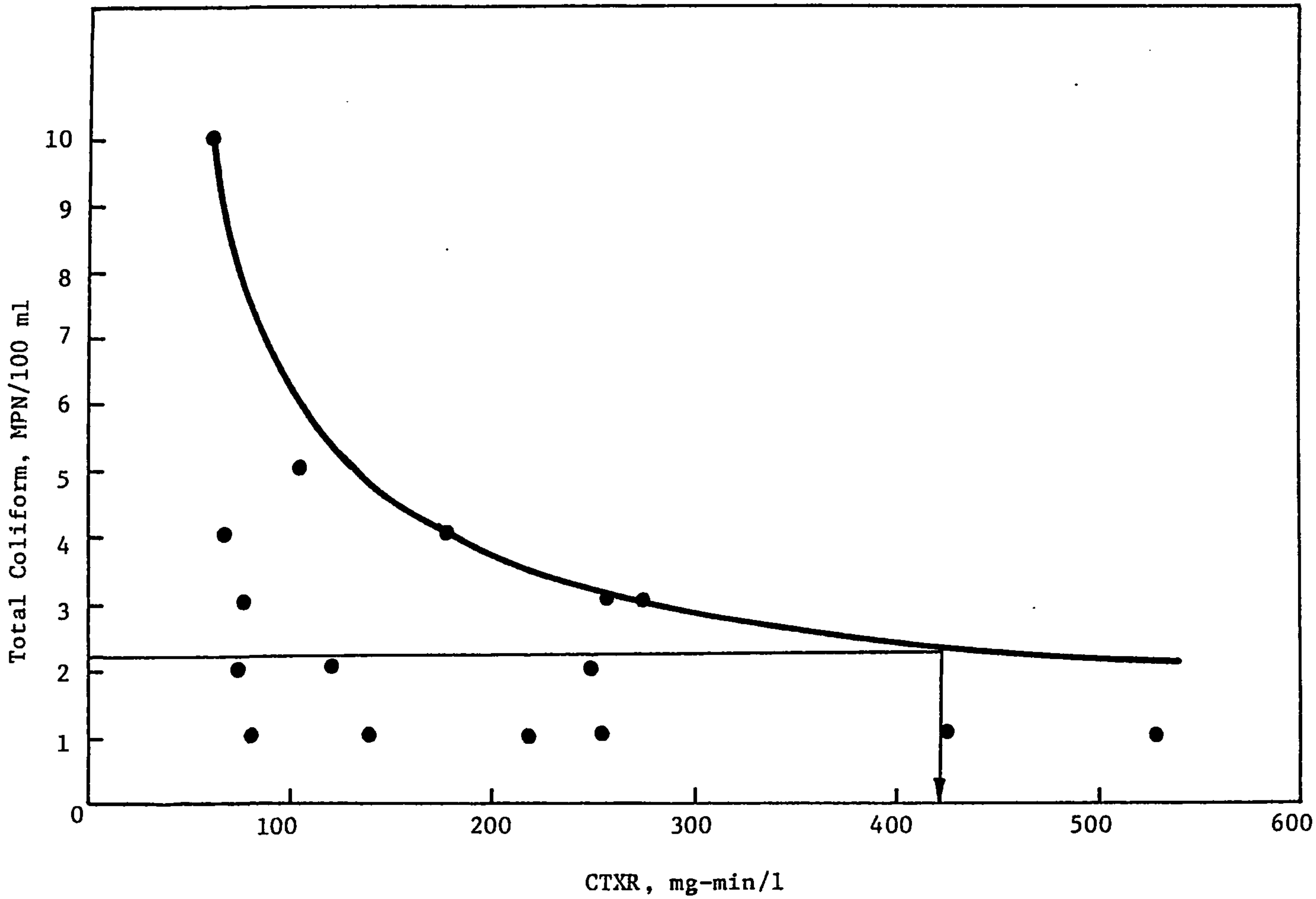

Fig. 5.16 : Total Coliform Count Versus Contact Time Multiplied by Chlorine Residual

case of Pomona Virus Study (12). The chlorination system employed in Pomona Virus Study was a continuous flow system while for this study it was a batch system. Fig. 5.17 shows the normalized residence time distribution in response to a pulse tracer input for the chlorination tank with a 40:1 length to width ratio as used in Pomona Virus Study. The initial time for dye detection is 0.43 of theoretical detention time. A batch system can be compared to a continuous flow system in the following manner. Keeping the concentration of residual chlorine constant, the value of contact time-residual chlorine for the batch process is divided by 0.43 in order to convert this value for a continuous system from a batch system. The calculated contact time-residual chlorine is 430/0.43 = 1000 and, therefore, the batch disinfection of this study yielded a result similar to the continuous flow system of Pomona Virus Study.

5.4 SYSTEM'S PERFORMANCE

The overall tertiary treatment system consisting of flocculation, sedimentation, filtration, and chlorination was very effective in achieving the required effluent quality, as described in Title 22 California Administrative Code, for both kinds of secondary effluents from North ARAMCO Wastewater Treatment Plant, Dhahran. Based on this bench scale study, the recommended design parameters for the individual processes are as follows :

Chemicals :

Alum as coagulant.

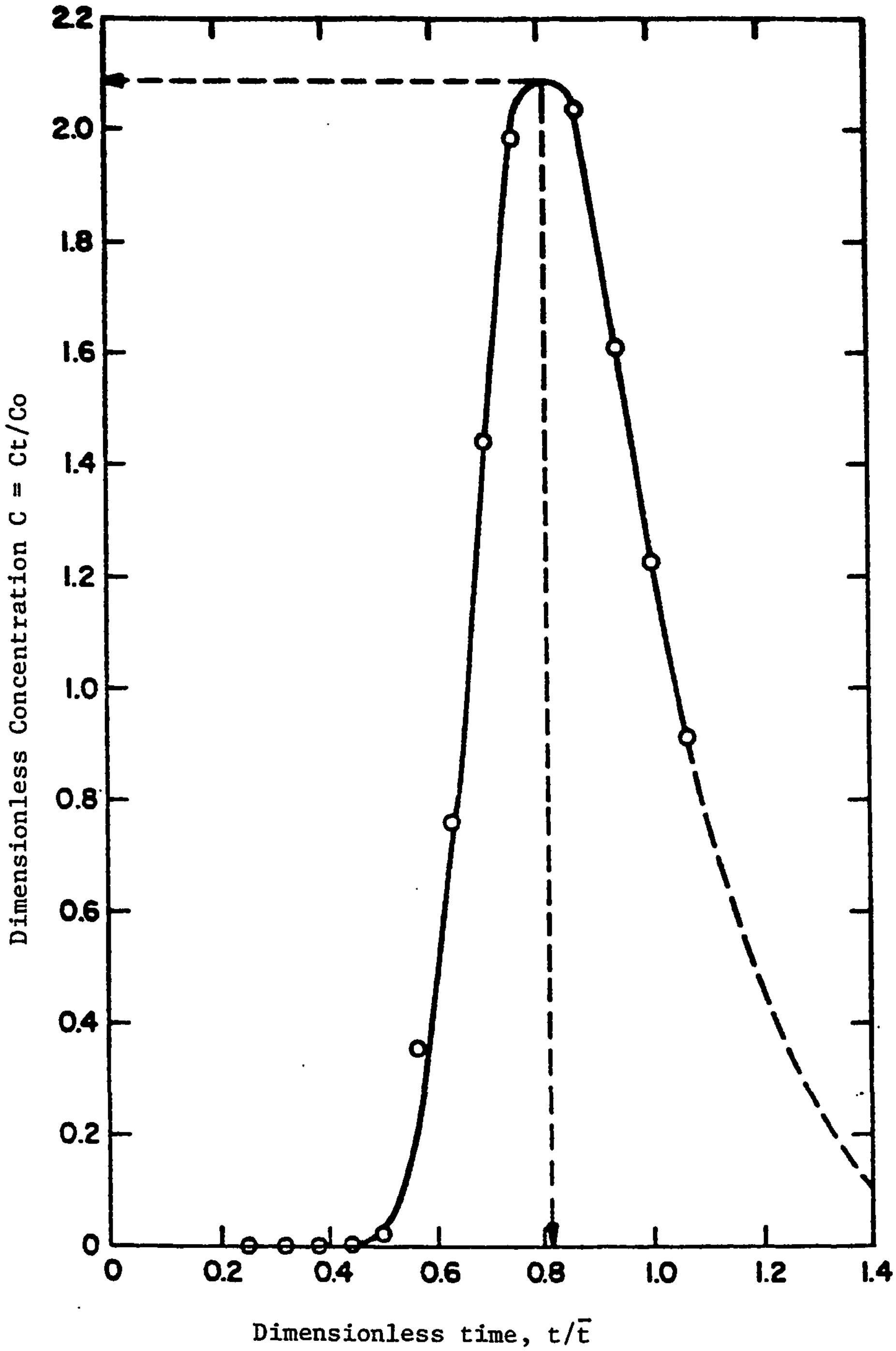

Fig. 5.17 : Normalized Residence Time Distribution in Response to a Pulse Tracer Input in a Chlorination Tank with 40:1 Length to Width Ratio (Adopted from 12)

Magnafloc 155 anionic polymer as a coagulant aid

Detention Times :

Rapid mix tank, 1 minute

Flocculation tank, 30 minutes

Sedimentation tank, 1.5 hours.

Filters :

Rapid gravity sand filter	
Depth of sand,	0.90 m
Effective size of sand,	1.25 mm
Uniformity coefficient of sand,	1.36
Nominal hydraulic loading,	2.7 $l/m^2.s$(4.0 gpm/ft^2)
Peak hydraulic loading,	4.1 $l/m^2.s$(6.0 gpm/ft^2)

Chlorination :

Contact time x chlorine residual,	1000 min-mg/l

Chapter 6

CONCLUSIONS

The following conclusions are drawn on the basis of the results obtained in this study :

1. The tertiary treatment system consisting of flocculation, sedimentation, filtration, and chlorination is an effective process for treating secondary effluents of North ARAMCO Wastewater Treatment Plant, Dhahran.
2. For unchlorinated secondary effluent, chemical dosage in the range of 5 to 10 mg/l alum with 0.1 to 0.3 mg/l polymer (Magnafloc 155) provided a satisfactory filtered effluent turbidity of less than 0.4 NTU. The optimum chemical dosage was 10 mg/l alum with 0.2 mg/l polymer based on the maximum turbidity removal and acceptable filter headloss.
3. For chlorinated secondary effluent, chemical dosage in the range of 5 to 10 mg/l with 0.2 to 0.3 mg/l polymer provided a satisfactory filtered effluent turbidity of less than 0.2 NTU. The optimum chemical dosage was 10 mg/l alum with 0.3 mg/l polymer based on the maximum turbidity removal and acceptable filter headloss.
4. During filter operation, the headloss was limited to 20 to 40 cm in 8 hours at a hydraulic loading of 2.7 $l/m^2.S$ (4.0 gpm/ft^2) and the effluent turbidity was at all times less than 0.4 NTU. At a filter hydraulic loading of 4.1 $l/m^2.s$ (6.0 gpm/ft^2), the headloss

increased significantly to greater than 80 cm in 8 hours, however, the effluent turbidity remained under 0.4 NTU. Therefore, the recommended normal filter hydraulic loading is 2.7 $l/m^2.s$ (4.0 gpm/ft^2) with a maximum filter hydraulic loading of 4.1 $l/m^2.s$ (6.0 gpm/ft^2) during peak flow hours.

5. The product of contact time and chlorine residual for batch disinfection to achieve a coliform concentration less than 2.2 per 100 ml was 430 min-mg/l for filtered unchlorinated effluent.

Chapter 7

RECOMMENDATIONS

Based on the present study, the following are recommendations for further research.

1. A pilot plant study can be performed with a larger diameter filter with a complete air-water backwash system and a continuous flow chlorination tank constructed at the North ARAMCO Wastewater Treatment Plant, Dhahran.

2. Test the pilot plant with a flocculation, sedimentation, filtration process as well as a direct filtration (without sedimentation) process.

3. Evaluate different gradations of sand media and filter depths to observe the performance of these media.

4. Conduct seeding experiments by introducing sufficient virus concentrations, or similar kinds of pathogenic organisms, to produce measurable effluent virus concentrations. The purpose would be to determine the removal efficiency of viruses by the system.

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APPENDIX A

The recommended standards for water works, GLUMRB (34) recommend that quick mixing should be performed with mechanical mixing devices, for rapid dispersion of chemical throughout the water, with a detention time not less than 30 seconds. The following recommendations are made for flocculation basins:

i) Inlet and outlet design shall prevent short-circuiting and destruction of floc.

ii) Minimum flow-through velocity shall not be less than 2.5 nor greater than 7.5 mm/s with a detention time for floc formation of, at least, 30 minutes.

iii) Agitators shall be driven by variable speed drives with the peripheral speed of paddles ranging from 0.15 to 0.75 m/s. Flocculation and sedimentation basins shall be as close together as possible.

Design of Flocculation Basin :

(a) *Observations*

Total volume of flocculation basin	= 10 litre
No. of chambers	= 3
Size of one flocculation chamber	= 15 cm x 15 cm x 15.6 cm
Size of one flocculation paddle	= 7.6 cm x 2.5 cm x 0.1 cm

(b) *Calculation of Mean Velocity Gradient, G*

Detention time, t = 30 min

Flow $Q = \frac{V}{t} = \frac{10 \times 1000}{30} = 335$ ml/min = 5.58 ml/s

Horizontal velocity $v = \frac{Q}{A} = \frac{5.58 \times 10}{15 \times 15.6} = 0.24$ mm/sec

No. of flocculator paddles per chamber = 2

Area of paddle = 2 x 7.6 x 2.5 = 38 cm²

which is 17% of total cross sectional area of flocculation chamber. Beam (35) recommends that in the flocculator basin paddles should not exceed 15-20% of the cross sectional area of the basin to prevent rolling of the water.

(i) Rotational Speed

$$V_P = \frac{2\pi \gamma n}{60}$$

where

V_P = velocity of paddle blades, cm/sec

n = number of revolutions per minute

r = distance from shaft to centre of paddle, cm

$$V_P = \frac{2\pi \times 3.8 \times 45}{60} = 17.91 \text{ cm/sec}$$

The velocity differential between paddles and fluid is therefore 0.70 x 17.91 = 12.53 cm/sec.

(ii) Total power input is determined as

$$P = C_D\ A.P.\frac{v^3}{2}$$

where

C_D = 1.2

A = paddle area = 38 cm^2

v = velocity differential - 12.53 cm/s

Then

$$P = 1.2 \times \frac{38}{10^4} \times 992.2 \times \frac{(12.53)^3}{10^6} = 8.9 \times 10^{-3}\ \text{N-m/s}$$

and

$$G = \left(\frac{P}{V\mu}\right)^{0.5}$$

where

V = tank volume = 15x15x15.6 = 3510 cm^3

μ = dynamic viscosity

At 40°C temperature, $\mu = 0.653 \times 10^{-3}$ kg/m.s

$$G = \left(\frac{8.9\times10^{-3}}{3510\times10^{-6}\times0.653\times10^{-3}}\right)^{0.5} = 62 \text{ per s.}$$

Values of G between 10 and 75 per second have been found to promote floc growth without destruction of the floc particle (36).

APPENDIX B

```
SJOB
C PROGRAM USED TO PLOT THE THEORETICAL CURVE FOR TRACER STUDY ON
C FLOCCULATION CHAMBER.
      REAL*4 MU(75),Y(100),MUI
      M=75
      DT=0.02
      NT=99
      DBYUL=0.34
      U=0.5*(1./DBYUL)
      T=0.00
      CALL TANP(M,U,MU)
      DO 99 I=1,M
      WRITE(6,101)I,MU(I)
  101 FORMAT(' ','I=',I3,' MUI=',F20.10)
   99 CONTINUE
C-----------------------------------------------------START  TIME LOOP
      DO 8 IT=1,NT
      T=T+DT
C------------------------------------- START SERIES SUM
      SUM=0.0
      DO 1 I=1,M
      MUI=MU(I)
      USQ=U*U+MUI*MUI
      EX=EXP(U-(USQ/2./U)*T)
      TERM=EX*MUI*(U*SIN(MUI)+MUI*COS(MUI))/(USQ+2.*U)
      SUM=SUM+TERM
      FAC=ABS(TERM/SUM)
      IF(FAC.LE.0.00001) GOTO 2
    1 CONTINUE
C------------------------------------- END SERIES SUM
      WRITE(6,20)
   20 FORMAT(' ','MORE TERMS NEEDED')
      STOP
    2 Y(IT)=2.*SUM
      WRITE(6,100)T,Y(IT)
    8 CONTINUE
  100 FORMAT(' ',2F15.10)
C100  FORMAT(' ',' T=',F10.5,' C/CO=',F10.5)
C-----------------------------------------------------END TIME LOOP
      STOP
      END
C-----------------------------------------
C     ***PROGRAM FOR OBTAINING THE ROOTS OF THE EQUATION *****
C                       A*TAN A = C
C              USING NEWTON RAPHSON METHOD
C-----------------------------------------

      SUBROUTINE TANP(M,U,Z)
      IMPLICIT REAL*8(A-H,O-Z)
      REAL*4 Z(M),SNGL,U
      UD=DBLE(U)
      PI=3.141526535 D 00
      DO 2 N=1,M
      X=PI*(N-1)+.01
      TOL=0.00001
C     WRITE(6,100)N,X
C 100 FORMAT(' ','FOR ROOT # ',I2,' STARTING VALUE=',F10.5)
      CALL NEWRAP(UD,X,TOL)
      Z(N)=SNGL(X)
    2 CONTINUE
```

```
      RETURN
      END
C--------------------------------------------
C     SUBROUTINE FOR THE NEWTON RAPHSON METHOD
C--------------------------------------------

      SUBROUTINE NEWRAP(UD,X,TOL)
      IMPLICIT REAL*8(A-H,O-Z)
    2 CALL FUNC(FX,DFX,DX,X,UD).
C     WRITE(6,100)X,FX,DFX,DX
C 100 FORMAT(' ','X=',F10.5,' FX=',F10.5,' DFX=',F10.5,' DX=',F10.5)
      X=X-DX
      ER=DABS(DX/X)
      IF(ER.LT.TOL) GO TO 1
      GO TO 2
    1 RETURN
      END
C
C--------------------------------------------
C     SUBROUTINE CONTAINING THE FUNCTION AND ITS DERIVATIVE
C

      SUBROUTINE FUNC(FX,DFX,DX,X,UD)
      IMPLICIT REAL*8(A-H,O-Z)
      FX=2/DTAN(X)-(X/UD-UD/X)
      DFX=-2./DSIN(X)/DSIN(X)-(1./UD+UD/X/X)
      DX=FX/DFX
      RETURN
      END

      SENTRY
I=   1 MUI=        1.5307260000
I=   2 MUI=        3.8681900000
I=   3 MUI=        6.7144140000
I=   4 MUI=        9.7249390000
I=   5 MUI=       12.7952300000
I=   6 MUI=       15.8925000000
I=   7 MUI=       19.0040100000
I=   8 MUI=       22.1238800000
I=   9 MUI=       25.2490800000
I=  10 MUI=       28.3778800000
I=  11 MUI=       31.5092000000
I=  12 MUI=       34.6423600000
I=  13 MUI=       37.7769100000
I=  14 MUI=       40.9125500000
I=  15 MUI=       44.0490400000
I=  16 MUI=       47.1861800000
I=  17 MUI=       50.3238900000
I=  18 MUI=       53.4620600000
I=  19 MUI=       56.6006100000
I=  20 MUI=       59.7394700000
I=  21 MUI=       62.8786100000
I=  22 MUI=       66.0179700000
I=  23 MUI=       69.1575400000
I=  24 MUI=       72.2973000000
I=  25 MUI=       75.4371900000
I=  26 MUI=       78.5772000000
I=  27 MUI=       81.7173000000
I=  28 MUI=       84.8576500000
I=  29 MUI=       87.9980000000
```

```
I= 30  MUI=        91.1384400000
I= 31  MUI=        94.2789600000
I= 32  MUI=        97.4195500000
I= 33  MUI=       100.5601000000
I= 34  MUI=       103.7009000000
I= 35  MUI=       106.8416000000
I= 36  MUI=       109.9824000000
I= 37  MUI=       113.1233000000
I= 38  MUI=       116.2642000000
I= 39  MUI=       119.4051000000
I= 40  MUI=       122.5460000000
I= 41  MUI=       125.6870000000
I= 42  MUI=       128.8280000000
I= 43  MUI=       131.9691000000
I= 44  MUI=       135.1102000000
I= 45  MUI=       138.2513000000
I= 46  MUI=       141.3924000000
I= 47  MUI=       144.5335000000
I= 48  MUI=       147.6747000000
I= 49  MUI=       150.8159000000
I= 50  MUI=       153.9571000000
I= 51  MUI=       157.0983000000
I= 52  MUI=       160.2395000000
I= 53  MUI=       163.3807000000
I= 54  MUI=       166.5220000000
I= 55  MUI=       169.6633000000
I= 56  MUI=       172.8045000000
I= 57  MUI=       175.9458000000
I= 58  MUI=       179.0871000000
I= 59  MUI=       182.2285000000
I= 60  MUI=       185.3698000000
I= 61  MUI=       188.5111000000
I= 62  MUI=       191.6521000000
I= 63  MUI=       194.7935000000
I= 64  MUI=       197.9349000000
I= 65  MUI=       201.0762000000
I= 66  MUI=       204.2176000000
I= 67  MUI=       207.3590000000
I= 68  MUI=       210.5004000000
I= 69  MUI=       213.6418000000
I= 70  MUI=       216.7832000000
I= 71  MUI=       219.9246000000
I= 72  MUI=       223.0660000000
I= 73  MUI=       226.2074000000
I= 74  MUI=       229.3488000000
I= 75  MUI=       232.4902000000
     0.0199999900   -0.0000092863
     0.0399999900   -0.0000043873
     0.0599999900    0.0001379043
     0.0799999800    0.0024641130
     0.0999999600    0.0132219000
     0.1199999000    0.0392059000
     0.1399999000    0.0831459[illegible]00
     0.1599999000    0.143258[illegible]000
     0.1799998000    0.21516[illegible]5000
     0.1999998000    0.293824300[illegible]
     0.2199998000    0.3746351000
     0.2399998000    0.4539459000
     0.2599998000    0.5291143000
     0.2799997000    0.5984072000
```

0.2999997000	0.6608077000
0.3199997000	0.7158541000
0.3399997000	0.7634695000
0.3599997000	0.8038476000
0.3799996000	0.8373565000
0.3999996000	0.8644642000
0.4199996000	0.8856958000
0.4399996000	0.9015929000
0.4599996000	0.9126944000
0.4799996000	0.9195176000
0.4999995000	0.9225512000
0.5199995000	0.9222517000
0.5399995000	0.9190362000
0.5599995000	0.9132859000
0.5799995000	0.9053469000
0.5999994000	0.8955298000
0.6199994000	0.8841133000
0.6399994000	0.8713461000
0.6599994000	0.8574512000
0.6799994000	0.8426214000
0.6999993000	0.8270348000
0.7199993000	0.8108419000
0.7399993000	0.7941780000
0.7599993000	0.7771631000
0.7799993000	0.7598987000
0.7999992000	0.7424780000
0.8199992000	0.7249784000
0.8399992000	0.7074679000
0.8599992000	0.6900069000
0.8799992000	0.6726454000
0.8999992000	0.6554285000
0.9199991000	0.6383929000
0.9399991000	0.6215696000
0.9599991000	0.6049865000
0.9799991000	0.5886639000
0.9999991000	0.5726221000
1.0199980000	0.5568758000
1.0399980000	0.5414361000
1.0599970000	0.5263125000
1.0799970000	0.5115115000
1.0999960000	0.4970389000
1.1199960000	0.4828972000
1.1399950000	0.4690883000
1.1599950000	0.4556118000
1.1799940000	0.4424678000
1.1999940000	0.4296542000
1.2199930000	0.4171677000
1.2399930000	0.4050058000
1.2599920000	0.3931642000
1.2799920000	0.3816380000
1.2999910000	0.3704222000
1.3199910000	0.3595126000
1.3399900000	0.3489024000
1.3599900000	0.338586[illegible]000
1.3799890000	0.32855[illegible]000
1.3999890000	0.3188117000
1.4199880000	0.3093415000
1.4399880000	0.3001400000
1.4599870000	0.2912018000
1.4799870000	0.2825204000

```
1.4999860000   0.2740893000
1.5199860000   0.2659025000
1.5399850000   0.2579535000
1.5599850000   0.2502360000
1.5799840000   0.2427440000
1.5999840000   0.2354720000
1.6199830000   0.2284135000
1.6399830000   0.2215628000
1.6599820000   0.2149144000
1.6799820000   0.2084624000
1.6999810000   0.2022017000
1.7199810000   0.1961263000
1.7399800000   0.1902315000
1.7599800000   0.1845122000
1.7799790000   0.1789630000
1.7999790000   0.1735794000
1.8199780000   0.1683565000
1.8399780000   0.1632895000
1.8599770000   0.1583738000
1.8799770000   0.1536054000
1.8999760000   0.1489796000
1.9199760000   0.1444925000
1.9399750000   0.1401398000
1.9599750000   0.1359177000
1.9799740000   0.1318223000

STATEMENTS EXECUTED=     7802

CORE USAGE        OBJECT CODE=     2904 BYTES,ARRAY AREA=      700 BYTES,

DIAGNOSTICS         NUMBER OF ERRORS=         0

COMPILE TIME=     0.04 SEC,EXECUTION TIME=      0.14 SEC,      13.44.19

         CSSTOP
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