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SURFACE WATER TREATMENT FOR COMMUNITIES IN DEVELOPING COUNTRIES

WASH TECHNICAL REPORT NO. 29

SEPTEMBER 1984

Prepared for:
Office of Health
Bureau for Science and Technology
U. S. Agency for International Development
Order of Technical Direction No. 89

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PREFACE

This volume is intended for planners and designers of water treatment plants to be built in Africa, Asia, and Latin America. In particular, the contents are addressed to treatment of surface waters for communities that, by virtue of being small or being located where supporting technical services are not readily available, should employ technologies that avoid the mechanization, instrumentation, and automation now common in the industrialized world.

Engineers educated in the industrialized world are taught to use technologies that are characterized as "capital-intensive." Their texts and references focus on the latest "high" technology which is marketed locally and supported with maintenance services and stocks of spare parts. These engineers are not familiar with technologies that minimize the need for support facilities. Such technologies are identified in this volume. Information concerning the performance of these technologies is provided where available. Otherwise they must be considered experimental. Readers who have experience with such technologies are urged to communicate their data to the authors (at the Department of Environmental Sciences and Engineering, University of North Carolina, Chapel Hill, North Carolina 27514, USA) for inclusion in later editions of this work. One purpose of this volume is to stimulate investigations into appropriate methods for surface water treatment.

Additional related information is available from many national and international agencies, and references to them appear in the text. Appendix E is a glossary of such organizations to which inquiries can be addressed.

The authors are indebted to engineers on the staffs of the Water and Sanitation for Health (WASH) project, the Office of Health of the US Agency for International Development, and Camp Dresser and McKee Inc. of Boston, Massachusetts for supporting this project; and to Ms. Phyllis Carlton of UNC for the painstaking effort involved in preparing the text for publication.

Special thanks are owed to engineers from Latin America who have been innovative in developing practices appropriate to the needs in their countries, practices which should be useful in other parts of the world. Engineers Jorge Arboleda Valencia, Jose Azevedo-Netto, Carlos Richter, Jose Perez, and Renato Pinheiro among many were particularly helpful. Felix Filho helped in the translation of the Spanish and Portuguese documents. Ann Jennings drew many of the figures in the manual. Herb Hudson and John Briscoe gave valuable assistance in reviewing the manuscript at various stages of preparation. Responsibility for the material in this manual rests solely with the authors.

I. INTRODUCTION

The design of water supply facilities for communities in developing countries should be based upon the proper application of current technology. The social and economic differences between the developed and the developing countries explain why conventional approaches for designing water systems in the industrialized countries are not appropriate in developing countries. In industrialized countries, water projects use capital-intensive designs with a high degree of mechanization and automation in order to reduce the need for labor, which is high in cost. The prevailing economies in developing countries, however, are labor-intensive. This implies that a facility that can be built and operated with local labor will likely be more economical and more easily operated than a facility utilizing extensive technology. An investment of about $600,000 in capital equipment would be warranted to replace an around-the-clock attendant in the United States based upon a total cost of $20,000 per year including fringe benefits, for each of the four persons required to provide continuous attendance, the 15-year life of the equipment, and the 10% interest rate (Okun, 1982). On the other hand, in a developing country, $20,000 might be the maximum investment warranted, based upon a wage of $1000 per year, 10-year equipment life, and a 20% interest rate. This difference is exacerbated when transportation costs and custom duties for imported equipment are considered. Moreover, the importation of mechanized equipment leaves the client in the developing country dependent on the foreign manufacturer for spare parts and maintenance skills which are not available locally.

The widespread, if inappropriate, use of sophisticated technologies in the developing countries can be readily explained:

1) The expatriate engineers employed in developing countries are generally familiar only with the technology espoused in the industrialized countries and are unfamiliar with the culture and competence of the people in the developing country;

2) The client in the developing country wants to appear up-to-date; therefore, he desires "only the best," which is erroneously translated to mean the latest, or the most complex technology;

3) The water treatment facilities are often the most expensive and visible of all investments made by communities in developing countries; therefore, the clients are more likely to opt for modern, sophisticated designs rather than for the use of simple technology.

Turnkey projects are also a major contributor to treatment plant failures in countries under development. They call for a single organization to take on the responsibility for planning, designing, constructing, and providing equipment for an entire water supply project. This approach gives rise to numerous disadvantages for developing countries, the most important being the propensity for the turnkey contractor to select capital-intensive designs because of their great profitability. The end result is community dependence on the turnkey contractor for spare parts and skilled maintenance, both of which are exceedingly expensive and incur slow delivery, so that facilities are often inoperative for years at a time.

Examples of Inappropriate Technology

The undesirable results of the implementation of inappropriate technologies are especially noticeable in the treatment units of water plants: coagulation and rapid mixing, flocculation, sedimentation, filtration, and disinfection. Several examples from the developing countries can be cited (Okun, 1982): (Okun, 1982):

*In a relatively new water treatment plant serving a capital city in Africa, extensive equipment and instrumentation have been installed. Despite the fact that the plant was only two years old, few of the instruments and none of the recorders were operative and much of the equipment was in poor condition. A representative of the expatriate consulting engineering organization was asked how this plant might have been designed differently had it been designed for his own city. After a moment of thought, he said with a touch of pride: "We did everything for this city that we would have done for ourselves." In his city the personnel were available to assure sound operation. Supporting services, particularly from the manufacturers of the equipment, were readily available by phone. In many developing countries, reaching for the phone promises the first frustration.

*Prior to World War II, the capital city of an Asian country was amply served with a conventional water treatment plant, including a low-cost horizontal-flow sedimentation basin built of concrete, conforming to the topography and not involving any mechanical equipment. On occasion, the tank was dewatered for sludge removal. Following the war, when the population of the city began to explode, a turnkey contractor was invited to increase the capacity of the plant. What was installed was a highly complex upflow unit made of steel with steel distributors and launders, all of which require extensive maintenance. Also, the unit was exceedingly difficult to operate. Here is a situation where local conditions should have helped dictate the most appropriate design. Upflow clarifiers are generally more economical than horizontal-flow tanks and are widely used in the industrialized world because they take up little space, require little manpower for their operation, and can provide excellent solids removal so long as their hydraulic capacity is not exceeded. In developing countries, on the other hand, horizontal-flow sedimentation tanks without mechanical sludge removal are to be preferred because they require no importation of equipment, and labor for cleaning the tanks is readily available. Space is generally not restricted. Most important, horizontal-flow tanks can be overloaded without serious detrimental impact on the subsequent filters, as most of the solids will still settle out. When upflow units are overloaded, however, sludge escapes from the blanket in large amounts and clogs the filters, interfering with the entire process. Water plans in developing countries are almost always overloaded, because capacity generally lags far behind demand.

*On a site for a new water treatment plant in a large city in Asia, large numbers of men and women with baskets on their shoulders were removing earth that had been hand excavated for the construction of the settling tanks. The local Asian contractor had decided correctly that it was more economical to use low-cost labor than to invest in excavating equipment. However, the design for the settling tank to go into that excavation called for the most modern mechanical sludge removal devices.

*At the same plant, when visitors asked to see how the filters were washed, three laborers were immediately available to turn the hand wheels on the valves for the filter influent, effluent, and wash water. In a neighboring large city, on the other hand, where labor is just as plentiful and low cost, the operating tables for the filters were all automatic and electrically operated, with push-button standbys, and the operation took place from a central control room.

*At a large new plant in Asia, a modern solids contact unit with mechanical sludge removal facilities was found to be out of operation. The mechanical equipment had never been operative. They had been waiting more than a year for a spare part from Europe, but in the meanwhile had been putting water through the unit. As it happens, the raw water was of such high quality that in a year's time virtually no sludge had accumulated. The investment in equipment was clearly unnecessary.

*At this plant, a sampling table had been installed in the laboratory that would permit pumping samples to be taken from any one of 96 points in the plant at the turn of a switch. However, only two samples were being tested each day. Furthermore, long sample lines distort sample quality. Better samples would have been obtained at lower cost, and more would have been done for the economy and the people if 96 persons had been employed for sample collection.

Accordingly, rather than transferring technology from the industrialized world to the developing world, engineers from the industrialized world might well learn something of the simplified practices that have been found satisfactory in the developing world so that, as they provide assistance to other countries of the developing world, they would be using technologies that are appropriate and that can be easily operated and maintained.

Purpose and Organization

In recent years much information has been disseminated on appropriate technologies in the water supply and sanitation fields, but most has been directed to facilities for individual households or groups of a few houses. the subject matter of this manual is the use of appropriate technologies for communities that require public water supplies as opposed to individual facilities. However, it is not concerned generally with water treatment in very large urban centers which have the resources and infrastructure to adopt mechanized water treatment facilities. The solutions that are proposed herein address the proper application of water treatment technologies in developing countries by advocating the design of treatment plants which are labor intensive, have low capital and recurrent costs and, by using indigenous resources, are tailored to the social and economic milieu of the region.

This manual is intended to be an aid to engineers designing new water plants or upgrading old ones in developing countries, as well as to government officials in these countries who need information concerning appropriate and economical water treatment. Moreover, this manual should enable planners and policy makers to take an initial step toward the development of simple design criteria and standard design manuals that are tailored for local conditions.

The intention of this manual is not to repeat information that is already well-documented in standard engineering works, but rather to focus on technologies that are not readily available in books or journals and, moreover, are not generally used in conventional water treatment practices. Of course, some types of conventional technologies in the industrialized countries are applicable in the developing countries; where such technologies are mentioned in this manual, references are made to appropriate sources for additional information. The selected bibliography at the end of the volume should be particularly valuable to users of this manual. It lists, in part, relevant books pertaining to water treatment in developing countries that have been published by the International Reference Center for Community Water Supply (IRC), the World Health Organization (WHO), the Pan American Health Organization (PAHO-CEPIS), and the German Agency for Technical Cooperation (GTZ). These publications can be readily obtained from the appropriate agency.

After a chapter (Chapter 2) on the basic considerations that must be addressed prior to the actual design of water treatment plants, the six chapters that follow (Chapters 3-8) present appropriate treatment requirements and processes for plants that are to be designed for communities in developing countries. A presentation of standardized designs, particularly those pertaining to package and modular-designed plants, is presented in Chapter 9. Chapter 10 reviews cost data for water treatment plants constructed in developing countries that may be useful for planning purposes. Chapter 11 examines the human resources needed to operate and maintain water treatment plants in developing countries and considers the requirements for the training of the required personnel. The more valuable and proven technologies are summarized at the end of this chapter.

Material on chemicals, hydraulic calculations, and simple methods for water analysis, together with a checklist for design are included in the appendices. A selected Bibliography and References conclude the manual.

The metric system of measurement, in units familiar to those working in the water supply field, is used predominantly in this manual. Common conversion factors for units used in the US are in Table 1-1.

Unless otherwise stated, costs in this manual have been adjusted to March 1982 United States dollars using the Engineering News Record (ENR) index; currency conversions have been made using July 1982 exchange rates. The procedure that has been utilized for adjusting costs is outlined in Chapter 10.

TABLE 1-1: Conversion to US Customary Units

TO CONVERT FROM:	UNITS	MULTIPLY BY:	TO OBTAIN:	UNITS
1) Flow				
- cubic meters per day	m^3/day	2.64×10^{-4}	million gallons per day	MGD
- cubic meters per second	m^3/day	22.8	milliion gallons per day	MGD
2) Overflow Rate				
- meters per day	m/d	9.91	gallons per square foot per day	gpcd
3) Water Demand				
- liters per capita per day	lpcd	0.26	gallons per capital per day	gpcd
4) Power				
- watt	W	1.34×10^{-3}	horsepower (water)	hp
5) Pressure				
- pasCal	Pa	1.45×10^{-4}	pound-force per square inch	psi

Summary

The following technologies are judged to be of merit in considering options for surface water treatment in communities in developing countries. Planners, managers, and engineers would do well to see that these technologies are among those that are evaluated before final selection of design approaches.

1) Pretreatment - Pretreatment refers to the "roughing" treatment processes such as plain sedimentation, storage, and roughing filtration, which are designed to remove the larger sized and settleable material before the water reaches the initial primary treatment units. Appropriate pretreatment during periods of excessive turbidity can reduce the load on subsequent treatment units and yield substantial savings in overall operating costs, especially for chemicals.

2) Chemicals - The chemicals necessary in water treatment include a coagulant, generally alum; disinfectants, generally chlorine or hypochlorites; and, when necessary, alkalies, generally lime, for pH control. Coagulant aids may be used to improve treatment and/or reduce coagulant consumption, with natural aids preferred over synthetic types.

3) Chemical feeders - Feeders should be simple in design and easy to operate. Hypochlorite and coagulant solutions may be fed by simple solution-type feeders that

can be constructed locally. Chlorine gas controllers are more complex than solution-type feeders; hence their use is limited to larger plants where skilled supervision is available. The use of saturation towers makes it possible to use inexpensive chemical compounds of low purity (e.g. lime or alum lumps) which may be available locally.

4) Hydraulic rapid mixers - Rapid mix units are located at the head end of the plant and are designed to generate intense turbulence in the incoming raw water. Hydraulic rapid mixers, such as hydraulic jumps, flumes, or weirs can achieve sufficient turbulence without the need for mechanical equipment and are easily constructed, operated, and maintained with local materials and personnel. The coagulant is added to the raw water by means of an above-water perforated trough or pipe diffuser and placed immediately upstream of the point of maximum turbulence.

5) Hydraulic flocculators - Flocculation follows directly after the rapid mix process, and provides gentle and continuous agitation during which suspended particles in the water coalesce into larger masses so that they may be removed from the water by subsequent treatment processes, particularly by sedimentation. Hydraulic flocculators, such as baffled-channel, gravel-bed, and heliocoidal-flow types, do not require mechanical equipment nor a continuous power supply, and can be built largely of concrete, brick, masonry, or wood with local labor at relatively low cost.

6) Horizontal-flow settling basins - The sedimentation process is responsible for the settling and removal of the suspended material from the water. Horizontal-flow basins with manual sludge removal, require no importation of equipment, and labor for cleaning the tanks is readily available. Equally important, horizontal-flow tanks can be overloaded without deleterious effects on subsequent filtration, as most of the suspended solids will still settle out. Inclined-plate or tube settlers may be installed in existing sedimentation basins to expand capacity and/or improve plant effluent quality.

7) Rapid filters - Filtration is a physical, chemical, and in some instances, biological, process for separating suspended and colloidal impurities from water by passage through porous media. A rapid filter consists of a layer of graded sand, or in some instances a layer of coarse filter media placed on top of a layer of sand, through which water is filtered downward at relatively high rates. The filter is cleaned by backwashing with water.

a) INTERFILTER-WASHING UNITS - Interfilter-washing filtration units working with declining rate are easier to build, operate and maintain than conventional rapid filters. Only two valves are needed for filter control, the entire system may be designed with concrete channels or box conduits, and it is possible to completely eliminate elaborate piping, valves, and controlling systems which are common to conventional filtration schemes.

b) DIRECT FILTRATION - The direct filtration process subjects the water to rapid mixing of coagulants, and sometimes flocculation, followed directly by filtration. Direct filtration is generally practicable only for raw waters that are low in turbidity, but it is a comparatively low-cost option when feasible, particularly in reducing the costly use of coagulants.

c) UPFLOW-DOWNFLOW FILTERS - In this type of system a battery of upflow roughing filters replaces the conventional arrangement for mixing, flocculation, and sedimentation used in rapid filtration plants. The downflow filter is a conventional rapid filter. This design can result in reduced construction and operational costs, the latter because the coagulant dosage is generally smaller than that used for the conventional treatment.

8) <u>Slow-sand filters</u> - A slow sand filter consists of a layer of sand through which water is filtered at a relatively low rate, the filter being cleaned by the periodic scraping of a thin layer of dirty sand from the surface at intervals of several weeks to months. Slow sand filters are effective in removing organic matter and microorganisms from raw waters of relatively low turbidity,

resulting in savings in disinfection. In addition, the cost of the construction of slow-sand filters in developing countries is low, the cost of importing the material and equipment is negligible, and the filters are easily constructed, operated, and maintained.

9) Modular water treatment plants - Modular plants are compact treatment units, generally made of concrete or masonry, and assembled either partly or entirely on-site without large or complicated equipment. Modular designs that are standardized reduce the type and number of plant components, thereby facilitating a more efficient system of procurement of spare parts, training of operators, and ease of repairs. To further shorten the time span for project implementation, plants may be comprised of modular units that are prefabricated, and easily transported to construction sites for final assembly.

II. BASIC CONSIDERATIONS

This chapter considers the principal factors upon which the appropriate selection of water treatment schemes is based. General design criteria are established for the implementation of water supply projects that reflect the prevailing social, economic, and technical conditions encountered in developing countries. Following this, the remaining sections of the chapter consider several important preliminary factors such as water quality criteria, choice of source, and choice of treatment processes, which should be investigated thoroughly before embarking on the design of treatment units. The individual unit processes are considered in subsequent chapters.

The selection of plant capacity, which is dependent upon many factors including population, design period, storage facilities, the distance between source and plant, and financial resources, are beyond the scope of this volume. Selection of the design period alone is no simple matter, depending as it does on rate of population growth, interest rates (which are a function of financial resources), the ease of expansion of the facilities, and the useful life of the component structures and equipment (Fair, Geyer, and Okun, 1971). Many text and reference books deal at considerable length with these issues. This manual analyzes the design of the treatment facilities after design capacity has been established.

General Design Guides for Practical Water Treatment

Design practice in any locality, whether it be in a developed or a developing country, should strive to optimize the total investment of available capital, material, and human resources, recognizing the limited resources of each that may exist. Inasmuch as socio-economic and technical conditions differ sharply between industrial and developing countries, a different set of design criteria should govern the implementation of water supply projects in each area.

In the industrialized countries, the prevailing capital-intensive economy has called upon the water supply industry to fulfill the following general conditions: (1) a high degree of automation in order to reduce labor costs which are substantially higher than those found in developing countries; (2) extensive utilization of equipment and instrumentation that is easily procured from and serviced by a variety of proprietors; and (3) preference for mechanical solutions rather than hydraulic ones. Treatment plants that have been designed under these conditions have performed reasonably well in the industrialized countries for decades, although in some instances, particularly in small communities, sophisticated plants that employ highly mechanized labor-saving equipment have often been shown to produce no real savings. Moreover, the reliability of the supply may not be increased, especially if adequate maintenance of such equipment cannot be assured. A common and unfortunate occurrence is the exportation of such design criteria, together with the equipment, to developing countries where they are entirely inappropriate, except perhaps in the very large urban centers where technical resources, support services, and qualified personnel are available.

Among the reasons why conventional technologies, such as those found in treatment plants in the US, are inappropriate overseas is that the capacity of the consumers in the developing world to pay for water is small, from 1/5 to 1/25 of that in the United States, so that plants constructed with expensive, imported technologies are not economically feasible (Wagner, 1982). Moreover, operation and maintenance costs, which are borne by the host country, increase proportionately with the complexity and sophistication of the treatment plant, resulting in higher water services charges for the consumer.

Second, there is a shortage of skilled personnel to operate and maintain treatment plants in the developing world; the limited numbers of qualified individuals are often attracted to the higher paying industries. On the other hand, there is an abundance of unskilled labor, which makes labor-intensive technologies more attractive.

Thirdly, the water utilities which must administer water systems in developing countries are generally weak and suffer from excessive staff turnover.

Accordingly, the following design guides are recommended for the design and construction of water treatment plants in developing countries (Arboleda, 1976; Wagner, 1982a):

1) To the extent possible, the utilization of mechanical equipment should be limited to that produced locally;

2) Hydraulically-based devices that use gravity to do such work as mixing, flocculation, and filter rate control are preferred over mechanized equipment;

3) Head loss should be conserved where possible;

4) Mechanization and automation are appropriate only where operations are not readily done manually, or where they greatly improve reliability;

5) Indigenous materials should be used to reduce costs and to bolster the local economy and expand industrial development;

6) For a variety of reasons (e.g. no fire demand, little lawn and garden watering), design estimates for per capita consumption and peak demands in the developing world should be much lower than those used in the US;

7) Design periods for construction should be made shorter to reduce the financial burden on the present population;

designs should be for 5 to 10 years rather than 15 to 20 years;

8) The plant must be designed to treat the raw water available. Because all waters are different, specific treatment objectives must be determined before initiating the design of plants.

The selection of water treatment methods that conform to the above-mentioned criteria does not require the creation of new technologies, but rather the innovative application of proven technologies. In some cases, it may be appropriate to use methods that were abandoned in the industrialized countries decades ago in favor of capital-intensive equipment (e.g. weir or hydraulic jump rapid mixers, baffled channel flocculators, solution-type feeders). Such simple technologies are readily adaptable to tailor-made treatment plant designs that are likely to provide more reliable service at lower cost to the community than those plants that are comprised largely of "shelf items" ordered from manufacturers abroad.

Water Quality Criteria

With the virtual disappearance of waterborne infectious diseases in the industrialized countries, more attention is being directed in those countries towards the public health effects of chronic diseases resulting from the presence of low concentrations of organic chemicals such as, for

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example, the chlorinated hydrocarbons (e.g. trihalomethanes) in drinking water supplies. The chronic effects of such chemicals require many decades of exposure before their impact can be discerned and so are not likely to be of importance where life-span is short and the relatively high incidence of waterborne infectious diseases such as typhoid and paratyphoid fevers, bacillary dysentery, cholera, and amoebic dysentery exact their toll, particularly as reflected in high infant mortality. Therefore, as enteric diseases are the predominant health hazard arising from drinking water in developing countries, standards for water quality should concentrate on microbiological quality. Furthermore, the removal of many chemical constituents from drinking water requires sophisticated treatment processes that are even beyond the technical and financial capabilities of most communities in the industrialized countries. In places where health-endangering chemicals are present in the water supply source, such as excessive nitrates (which can cause pediatric cyanosis) or excessive fluorides (which can cause bone diseases), it is preferable to change the source, if at all possible, rather than to provide sophisticated treatment.

A safe and potable drinking water should conform to the following water quality characteristics (IRC, 1981b). It should be:

1) Free from pathogenic organisms;

2) Low in concentrations of compounds that are acutely toxic or that have serious long-term effects, such as lead;

3) Clear;

4) Not saline (salty);

5) Free of compounds that cause an offensive taste or smell; and

6) Non-corrosive, nor should it cause encrustation of piping or staining of clothes.

In order to assure that such levels of water quality are maintained, many developing countries have established national standards for water quality adapted from the World Health Organization's International Standards for Drinking Water (WHO, 1971). Table 2-1 presents a comparison of physical and chemical guidelines for treated drinking water among those recommended by the WHO, the US, and several developing countries. The chemical compounds and water quality parameters that are of most concern in developing countries include iron and manganese, fluorides, nitrates, turbidity, and color. Similarly, guidelines for bacteriological water quality in the distribution system are compared in Table 2-2.

Choice of Source

The selection of the source determines the adequacy, reliability, and quality of the water supply. The raw water

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TABLE 2-1: Comparison of Chemical and Physical Drinking Water Standards Recommended by the WHO, USA and Several Developing Countries

CHEMICAL AND PHYSICAL STANDARDS	WHO RECOMMENDED STANDARDS	USA 1975 INTERIM[a] USA 1962[B]	INDIA (1973)	INDIA RECOMMENDED (1975)	KOREA	PHILIPPINES (1963)	QUATAR	TANZANIA (TEMP.) (1974)	THAILAND
Total hardness (meq/l) 1 meq/l = 50 mg/l as $CaCO_3$	2-10		12	12	6			12	6
Turbidity (NTU)	25	1-5[a]			2		5	30	5
Color (platinum-cobalt scale)	50				2		20	50	20
Iron, as Fe (mg/l)	1	0.3[b]	1	1	0.3	1	0.3	1	0.5
Manganese, as Mn (mg/l)	0.5	0.05[b]	0.5	0.5	0.3	0.5	0.3	0.5	0.3
pH	6.5-9.2		6.5-9.2	6.5-9.2		6.5-9.2		6.5-9.2	6.5-8.5
Nitrate, as NO_3 (mg/l)	45	45[a]	50	45	45	50		100	45
Sulfate, as SO_4 (mg/l)	400		400	400	200	400	250	600	250
Fluoride, as F^- (mg/l)	0.6-1.7	1.4-2.4[a]	2.0	1.5	1.0	1-1.5	1.6	8.0	1-1.5
Chloride, as Cl (mg/l)	600	250[b]	1000	1000	150	600	250	800	330
Arsenic, as As (mg/l)	0.05	0.05[a]	0.2	0.05	0.05	0.2		0.05	0.05
Cadmium, as Cd (mg/l)	0.01	0.01[a]		0.01		0.01		0.05	
Chromium (mg/l)	0.05	0.05[a]	0.05	0.05	0.05	0.05	0.05	0.05	0.05
Cyanide, as Cn (mg/l)	0.05	0.01[b]	0.01	0.05		0.01		0.2	0.2
Copper, as Cu (mg/l)	1.5	1.0[a]	3.0	1.5	1.0	1.5	0.3	3.0	1.0
Lead, as Pb (mg/l)	0.1	0.05[a]	0.1	0.1	0.1	0.1	0.1	0.1	0.05
Magnesium, as Mg (mg/l)	150		150	150		150	125		125
Mercury, as Hg (mg/l)	0.001	0.002[a]		0.001					
Selenium, Se (mg/l)	0.01	0.01[a]	0.05	0.01		0.05		0.05	0.01

[SOURCE: adapted from World Bank, 1977]

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TABLE 2-2: Comparison of Bacteriological Drinking Water Standards Recommended by the WHO, USA, and Several Developing Countries

WHO Recommended Standards (International, 3rd Edition)	1. Water entering distribution system; chlorinated or otherwise disinfected samples - 0/100 mg/l; non-disinfected supplies E. coli 0/100 ml; coliform 3/100 ml occasionally. 2. Water in distribution system: 95% of samples in a year - 0/100 ml coliform; E coli - 0/100 ml in all samples; no sample greater than 10 coliform/100 ml; coliform not detectable in 100 ml of any two successive samples. 3. Individual or small community supplies: less than 10/100 ml coliform; 0/100 ml E. coli in repeated samles.
USA	Coliform shall not be present in (a) more than 60% of the portions in any month; (b) five portions in more than one sample when less than five are examined/month; or (c) five portions in more than 20% of the samples when five or more samples examined/month.
India (1973)	Coliform = 0-1.0/100 ml permissive; 10-100/100 ml excessive but tolerated in absence of alternative, better source; 8-10/100 ml acceptable only if not in successive samples; 10% of monthly samples can exceed 1/100 ml.
India Recommended (1975)	E. coli = 0/100 ml. Coliform = 10/100 ml in any sample, but not detectable in 100 ml of any two consecutive samples or more than 50% of samples collected for the year.
Philippines (1963)	Coliform - not more than 10% of 10 ml portions examined shall be positive in any month. Three or more positive 10 ml portions shall not be allowed in two consecutive samples; in more than one sample per month when less than 20 samples examined; or in more than 5% of the samples when 20 are examined per month.
Quater	Coliforms 0/100 ml if present in two successive 100 ml samples, give grounds for rejection of supply.
Tanzania (Temporary 1974)	Non-chlorinated pipd supplies: 0/100 ml coliform - classified as excellent; 1-3/100 ml coliform - classified as satisfactory; 4-10/100 ml coliform - classified as suspicious; 10/100 ml coliform - classified as unsatisfactory; one or more E. coli/100 ml classified as unsatisfactory. Other supplies: WHO standards to be aimed at.
Thailand	Coliform = 2.2/100 ml. E. coli = 0/100 ml.

[SOURCE: World Bank, 1977]

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quality dictates the treatment requirements. For example, most groundwaters that are free from objectionable mineralization are both safe and potable, and may be used without treatment, provided the wells or springs are properly located and protected. Surface waters, on the other hand, are exposed to direct pollution, and treatment is usually a prerequisite for their development as a drinking water supply. The location of the source also defines the energy requirements for raw water pumping, which can directly affect recurrent operational costs.

Whenever possible, the raw water source of highest quality economically available should be selected, provided that its capacity is adequate to furnish the water supply needs of the community. The careful selection of the source and its protection are the most important measures for preventing the spread of waterborne enteric diseases in developing countries. Dependence upon treatment alone to assure safe drinking water in developing countries is inappropriate, because of inadequate resources, as illustrated by the poor record in these countries, for properly operating and maintaining water treatment plants, particularly with respect to adequate disinfection before the treated water enters the distribution system (NEERI, 1971).

Accordingly, groundwater is the preferred choice for community water supplies, as it generally does not require extensive treatment, and operation is limited to pumping and possibly chlorination. When not available from a natural source, groundwater can often be obtained by artificial recharge. In the event that no suitable aquifers are available, relatively clear waters from lakes or streams are preferred as these can be treated by slow-sand filtration. In the event that river waters are heavily silted, pretreatment may be provided by plain sedimentation or roughing filters prior to slow sand filtration. Only as a last resort should sources be developed that require chemical coagulation, rapid filtration, and disinfection. Even then, only simple, practical technologies such as gravity chemical feed with solutions, hydraulic rapid mixing and flocculation, horizontal-flow sedimentation, and manually operated filters should be used.

A sanitary survey of the potential drinking water sources for a community is an essential step in source selection. The survey should be conducted in sufficient detail to determine (1) the suitability of each source, based upon its adequacy, reliability, and its actual and potential for contamination; and (2) the treatment required before the water can be considered acceptable. Physical, bacteriological, and chemical analyses can, in addition, be helpful in providing useful information about the source and the conditions under which it will be developed. Guidelines for sanitary surveys are given in the WHO monograph _Surveillance of Drinking Water Quality_ (1976).

Choice of Treatment Processes

The broad choices available in water treatment make it possible to produce virtually any desired quality of finished water from any but the most polluted sources; therefore, economic and operational considerations become the limiting constraints in selection of treatment units. A treatment plant may consist of many processes, including pretreatment, chemical coagulation, rapid mixing, flocculation, sedimentation, filtration, and disinfection; which are arranged, in general, as shown in Figure 2-1. However, water quality varies from place to place and, in any one place, from season to season, and the resources for construction and operation vary from place to place, so the treatment selected must be based on the particular situation.

The primary factors influencing the selection of treatment processes are (Lewis, 1980):

1) treated water specifications;

2) raw water quality and its variations;

3) local constraints;

4) relative costs of different treatment processes.

These factors are discussed below.

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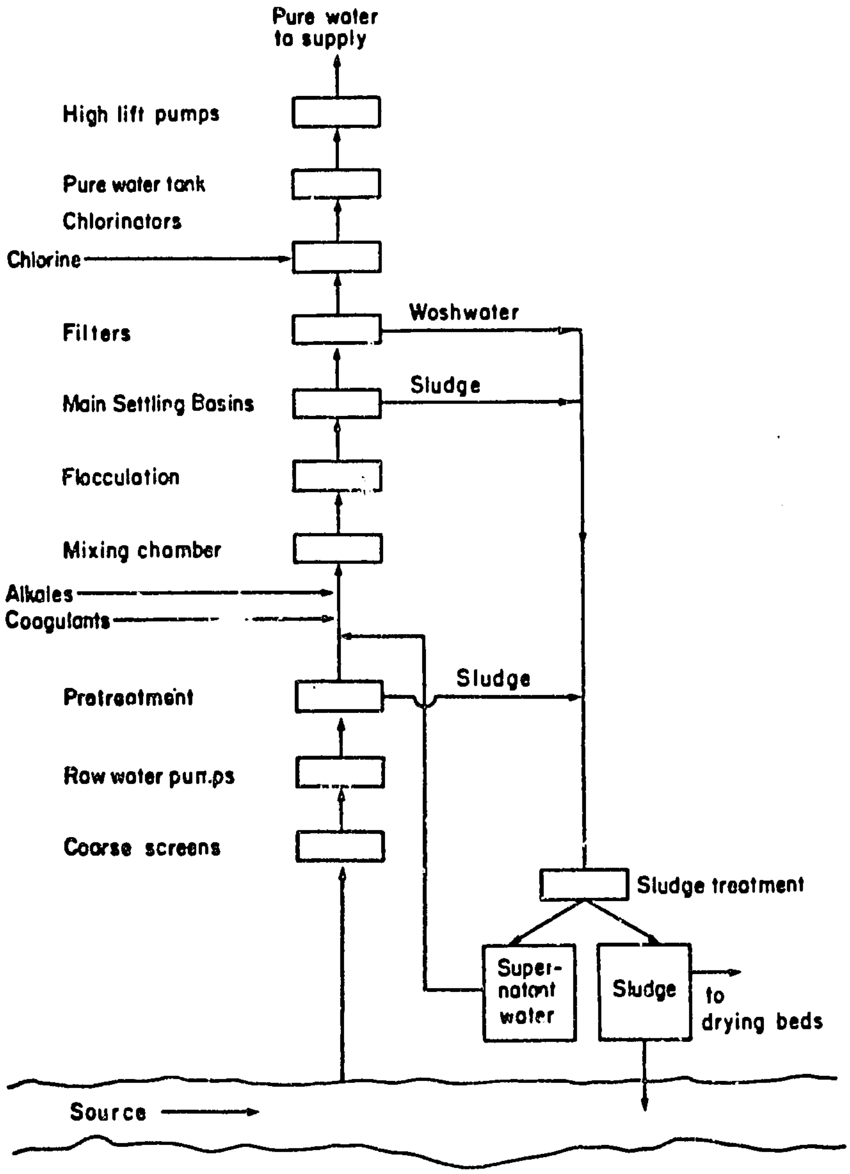
FIGURE 2-1

Flow Diagram Showing Possible Treatment Stages
in a Conventional Rapid Filtration Plant



[SOURCE: adapted from Smethurst, 1979, p. 19]

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Finished water requirements and raw water quality generally exert the greatest influence on process selection. Finished water specifications, as prescribed by the WHO, are presented in Tables 2-1 and 2-2; while Table 2-3 indicates the treatment necessary for raw waters of a variety of bacteriological and physical-chemical characteristics.

Local constraints that govern the implementation of water supply projects in developing countries, as discussed previously, are quite different from those of the industrialized countries. Considerations that local engineers or water supply planners must evaluate include:

1) Limitations of capital;

2) Availability of skilled and unskilled labor;

3) Availability of major equipment items, construction materials, and water treatment chemicals;

4) Applicability of local codes, drinking water standards, and specifications for materials;

5) Influence of local traditions, customs, and cultural standards; and

6) Influence of national sanitation and pollution policies.

The selection of appropriate treatment processes is facilitated by field and laboratory investigations. A sanitary survey that identifies sources of pollution and can help characterize raw water quality during dry and wet seasons is essential. Raw water analyses are helpful but,

TABLE 2-3: Classification of Raw Waters with Regard to Treatment Processes

CLASSES	MPN	TURBIDITY (NTU)	COLOR (color units)	IRON (mg/l)	TOTAL SOLIDS (mg/l)	CHLORIDES (mg/l)	HARDNESS (mg/l)	PLANKTON AND ALGAL GROWTH
I	<2.2	<25	<50	<1.0	<1500	<600	<250	insign.
II	<2.2 <50	<25	<50	<1.0	<1500	<600	<250	insign.
III	<2.2 <50	<25	<50	<1.0	<1,500	<600	<250	excess.
IV	<50	<25	<50	>1.0	<1,500	<600	<250	insign.
V	<50	<25	<50	<1.0	<1,500	<600	<250	insign.
VI	<1,000	<25	<70	<2.5	<1,500	<600	<250	insign.
VII	<5000	<75	--	<2.5	<1,500	<600	<250	insign.
VIII	<20,000	<250	--	<2.5	<1,500	<600	<250	insign.
IX	<20,000	<250	--	>2.5	<1,500	<600	<250	insign.
X	<20,000	<250	--	<2.5	<1,500	<600	<250	insign.
XI	<20,000	<250	--	<2.5	<1,500	<600	>250	insign.

* * * * *

CLASSES	MINIMUM TREATMENT POSSIBLE	SOURCE
I	Not necessary	protected spring
II	Chlorination	
III	Chemical pretreatment and chlorination	impounded reservoir
IV	Iron removal and chlorination	groundwater
V	Hardness reduction and chlorination	
VI	Slow-sand filtration and chlorination; superfiltration and chlorination	
VII	Upflow filtration and chlorination; superfiltration and chlorination	clear water from lakes or reservoirs
VIII	Coagulatoin-sedimentation-filtration-chlorination Superfiltration and chlorination	surface water
IX	Aeration-coagulation-sedimentation-filtration-chlorination	surface water
X	Pretreatment-coagulation-sedimentation-filtration-chlorination	very turbid rivers
XI	Coagulation-sedimentation-filtration-hardness reduction-chlorination	surface water

[SOURCE: adapted from Azevedo-Netto, 1982, personal communication]

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unless taken at all seasons, may be misleading as seasonal variations in raw water quality are often extreme in countries with well-defined rainy seasons. In some instances, it is possible to select designs based on experience in other plants treating water of similar quality, especially if this water derives from similar catchment areas in the same geographical region. For chemical coagulation, laboratory jar tests can be used to assess the optimum pH, the type and range of dose of primary coagulant, and the suitability of using coagulant aids (procedures for jar testing are covered in Chapter 4). Pilot plant studies are useful for evaluating design parameters for filtration processes and to a lesser extent sedimentation processes, but should be conducted through a long enough period so that sufficient information is generated under the entire range of expected operating conditions; although such studies need not be run every day or every week. Sedimentation cannot be reproduced accurately on a small scale due to the effects of density currents and wind action on full scale settling tanks (Chapter 7). Filtration, on the other hand, scales up readily and pilot plant results can be used directly to determine filter run lengths, filtered water quality, and type, depth, and size of the filter media. Moreover, such studies are helpful in deciding whether direct filtration is feasible or if conventional treatment must be used (Chapter

8, "Direct Filtration"). Pilot plant filter testing, and jar testing and evaluation are covered fully by Hudson (1981). Simple procedures for conducting such tests for sedimentation, slow-sand filtration, and rapid filtration are covered in the IRC publication Small Community Water Supplies (IRC, 1981b).

Records of all such studies, sanitary surveys, raw water analyses, jar tests, and pilot plant investigations, should be kept, as the accumulation of experience in a region can be the best guide to the planning of new water treatment schemes.

III. PRETREATMENT

For a variety of reasons, rivers in Asia, Africa, and Latin America tend to exhibit wide fluctuations in water quality, and, particularly, high turbidities in rainy seasons. Appropriate pretreatment during periods of excessive turbidity may reduce the load on subsequent treatment units and yield substantial savings on overall operating costs, especially for chemicals. In this manual, pretreatment refers to the "roughing" treatment processes designed to remove the larger-sized and settleable material from raw water before the water reaches the initial primary treatment units; i.e. chemical coagulation and mixing-in rapid filtration plants, or slow-sand filtration. In most cases, pretreatment is only justified for treating waters from turbid rivers or streams, as lakes, surface reservoirs, and other quiescent bodies of water inherently provide natural settling of the heavier suspended material. Furthermore, the seasonal variations of raw water quality in the rivers may make pretreatment necessary only during part of the year, such as during seasonal flooding. For other times of the year, the pretreatment units can be bypassed.

Proper location and design of intakes can minimize the requirements for pretreatment and protect treatment units. Where streams carry silt, the heavier metal tends to move along the bottom during periods of high flow; accordingly intake pipes should be located well above the bottom. In streams that vary significantly in level, it may be necessary to have intakes at different elevations in the stream with the lower intake being used for dry periods, and the higher intakes being used for wet periods. A box intake structure, with the inlet facing downstream, can be installed with the use of stop logs or planks to permit skimming water from the upper levels of a stream regardless of its depth. Bars can be placed over intakes to exclude debris. Sometimes they are mounted in frames which are duplicated so that one frame can be lifted for cleaning or repair without allowing unscreened water to the plant.

Table 3-1 indicates the usual conventional methods of pretreatment for given turbidities. The turbidity ranges for each of these methods are suggested by Huisman and Wood (1974) for pretreatment prior to slow-sand filtration; and are used here to serve as guidelines.

Pretreatment improves the performance of the unit processes in a rapid filtration plant: (1) better operation of the unit processes because raw water quality is the less variable; (2) less voluminous sludge is produced, and therefore less cleaning is needed for the main sedimentation basins; and (3) because a large portion of the suspended solids is removed in the pretreatment step, fewer chemicals are used in subsequent treatment.

TABLE 3-1: Conventional Methods of Pretreatment

TURBIDITY RANGE NTU[a]	PRETREATMENT
20 - 100	Plain sedimentation
>1000	Storage
20 - 150	Roughing filtration
50 - 200	Chemical pretreatment

a The nephelometric turbidity unit (NTU), the Jackson turbidity unit (JTU), and the formazin turbidity unit (FTU) are all numerically the same, and are interchangeable for all practical waterworks purposes.

For slow-sand filtration, pretreatment is essential if the raw water has a value of more than 50 NTU for periods longer than a few weeks or values above 100 NTU for more than a few days (Huisman and Wood, 1974). In fact, the best purification occurs when the average turbidity of the water on top of the slow-sand filters is 10 NTU or less.

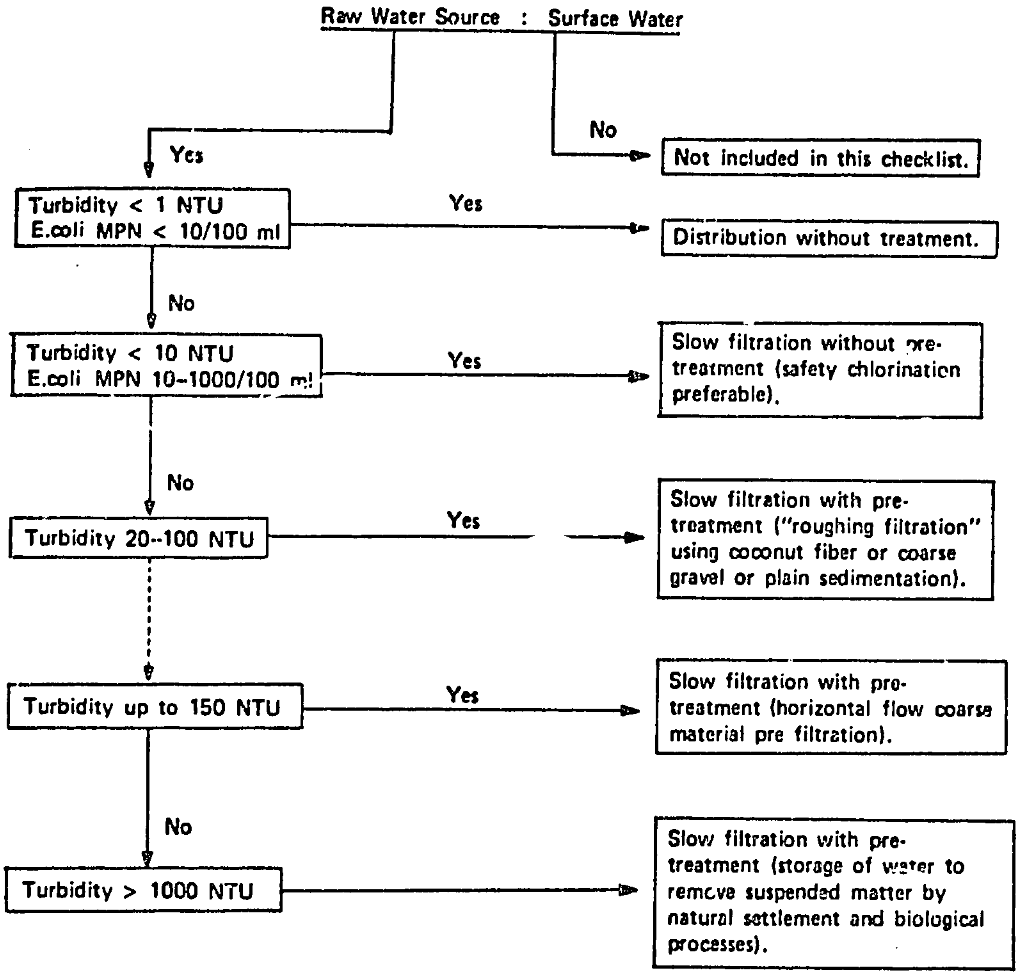
The selection of the most suitable type of pretreatment for a particular design should be made on the basis of field investigations in which samples are taken from all regimes of the river to determine variations in raw water characteristics. On the basis of two such characteristics, viz. Turbidity and E. coli content, the checklist presented in Figure 3-1 allows one to select appropriate pretreatment for slow-sand filtration (Thanh and Hetteratchi, 1982).

Plain Sedimentation

The process of plain sedimentation allows for the removal of suspended solids in the raw water by gravity and the natural aggregation of the particles in a basin, without the use of coagulants. The efficiency of this process, as measured by turbidity removal, is largely dependent on the size of the suspended particles and their settling rate. Table 3-2 shows particle diameters and settling ranges for suspended materials found in water. It is obvious from the data in Table 3-2 that plain sedimentation would serve no

FIGURE 3-1

A Checklist for the Selection of a Pretreatment Method to Supplement Slow Sand Filtration



[SOURCE: Thanh and Hettiaratchi, 1982, p. 14]

TABLE 3-2: Effect of Decreasing Size of Spheres on Settling Rate

Diam. of Particle, mm	Order of Size	Total Surface Area[a]	Time Required to Settle[b]
10	gravel	3.14 sq cm	0.3 sec
1	coarse sand	31.4 sq cm	3 sec
0.1	fine sand	314 sq cm	38 sec
0.01	silt	.314 sq meters	33 min
0.001	bacteria	3.14 sq meters	55 hr
0.0001	colloidal particles	31.4 sq meters	230 days
0.00001	colloidal particles	0.283 hectares	6.3 yr
0.000001	colloidal particles	2.83 hectares	63 yr minimum

[a] Area for particles of indicated size produced from a particle 10 mm in diameter with a specific gravity of 2.65.

[b] Calculations based on sphere with a specific gravity of 2.65 to settle 30 cm.

[SOURCE: adapted from AWWA (1971), p. 70] practical purpose for the removal of material smaller than 0.01 mm.

Plain sedimentation is quite effective in tropical developing countries for the following reasons: (1) the turbidity in rivers can be attributed largely to soil erosion, the silt being settleable; and (2) the higher temperatures in these countries improve the sedimentation process by lowering the viscosity of the water. Experience has shown that waters of high turbidity are more effectively clarified than waters of low turbidity. Plain sedimentation (or presettling) basins can be used as pretreatment units for both rapid and slow-sand filtration plants. In the latter case, however, its use is limited to where it is possible to reduce the raw water turbidity to 30 NTU or less to avoid too frequent clogging of the sand bed. The economic and technical feasibility of achieving such a limit using plain sedimentation may be determined from settling tests of the raw water (IRC, 1981b).

The design of plain sedimentation basins is similar to that of conventional settling basins, except that the detention times are shorter and the surface loadings are higher. The minimum depth of the basin is also somewhat less, as the sludge storage requirements are not as great as that in conventional basins which follow coagulation and flocculation. The basin may operate on a batch basis, being held empty until needed.

Practical experience by Smethurst (1979) in Baghdad and elsewhere confirms that effluents with considerably less than 1000 mg/l of suspended solids can safely be withdrawn from presettling tanks of one hour detention, even though the incoming water might have suspended solids of 10,000 mg/l or more. The design criteria for rectangular plain sedimentation basins are summarized in Table 3-3. The values listed are generalized, and serve only as guidelines. Chapter 7 contains additional information on sedimentation basin design and construction.

The construction of plain sedimentation basins can be quite simple. Three types of such basins are shown in Figures 3-2 to 3-4. The first type is constructed with wooden sheet piles, the second type is dug out of the earth with sloping sides, and the third type consists of a triangular shape with variable depth for achieving a uniform distribution of water both at the inlet and in the settling unit. Also, the design in Figure 3-3 has a slotted brick wall at the inlet side to uniformly distribute the flow, and a bypass channel to be used when presedimentation is not necessary or when the unit must be cleaned. Earthen basins may have to be lined with plastic, or an impermeable layer of clay, masonry, or concrete in places where seepage occurs, and protected from flooding. The addition of overflow weirs or baffles across the width of the basin can improve the uniform distribution of velocities and mitigate

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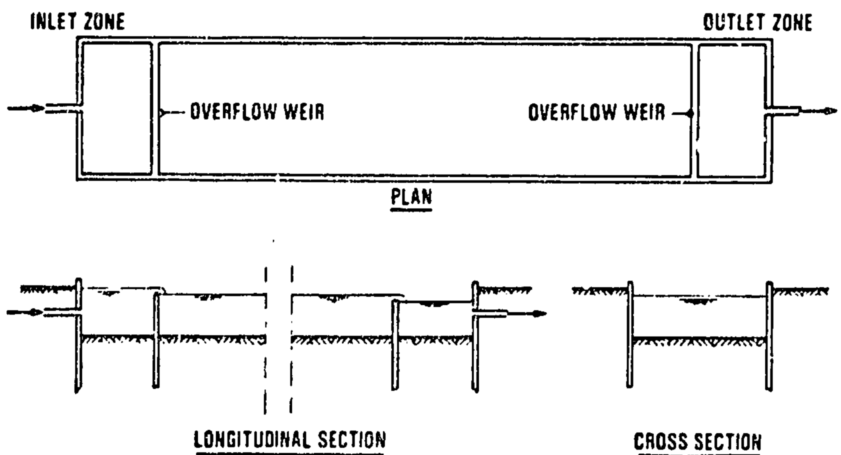
FIGURE 3-2

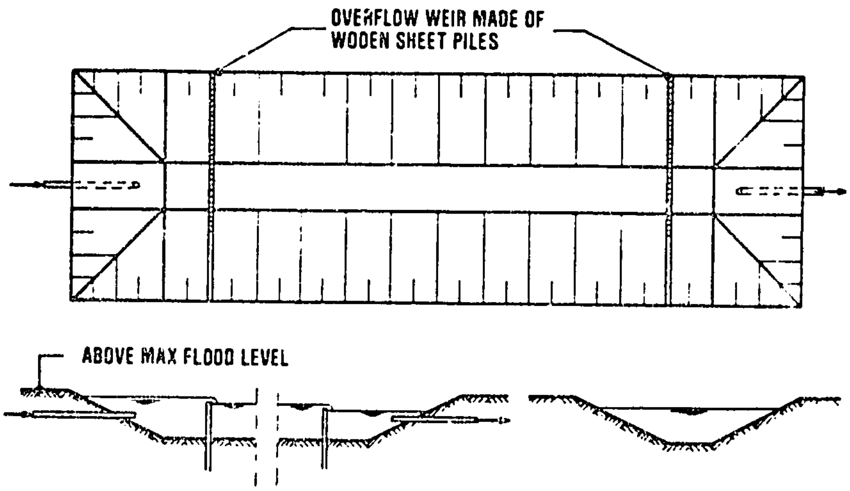
Presettling Basin Constructed with Wooden Sheet Piles



[SOURCE: IRC, 1981b, p. 236]

FIGURE 3-3

Dug Basin as a Presettling Tank

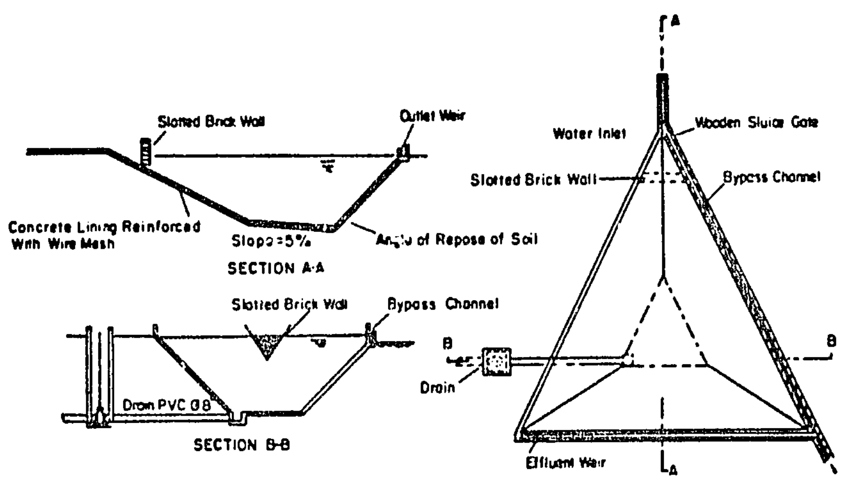


[SOURCE: IRC, 1981b, p. 236]

qV

FIGURE 3-4

Triangular Presettling Basin with Variable Depth



[SOURCE: CEPIS, 1982, vol. 2, plan no. 4]

TABLE 3-3: Design Criteria for Plain Sedimentation Basins

PARAMETER	RANGE OF VALUES	SYMBOL
Detention time (h)	0.5 - 3	V/Q
Surface loading (m/day)	60 - 80	Q/(L)(W)
Weir overflow rate (m^3/m per day)	60 - 80	Q/R
Depth of the basin (m)	1.5 - 2.5	H
Length/width ratio	4:1 - 6:1	L/W
Length/depth ratio	5:1 - 20:1	L/H

where: L = length (m)

W = width (m)

V = volume of the basin (m^3) = L x W x H

Q = flow rate (m^3/day)

R = total length of overflow of the outlet weir (m)

[SOURCE: adapted from Thanh et al. (1982), p. 24]

TABLE 3-4: Turbidity Removal with Different Settling Times (Mosul, Iraq)

Initial Turbidity (NTU)	TURBIDITY REMAINING (NTU) After 2 hrs	After 3 hrs
500	145	90
1,200	620	120
1,800	450	90
2,500	610	120

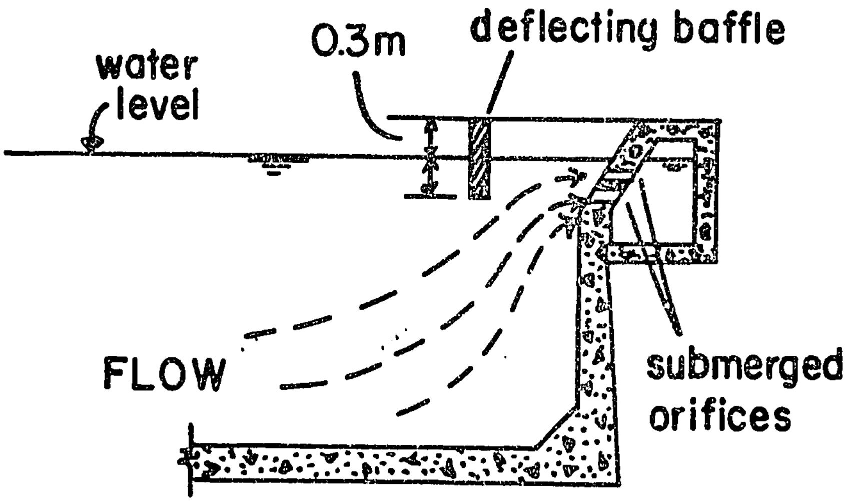
[SOURCE: Ahmad, Wais and Agha, 1982, p. 442] short-circuiting. In areas where floating algal growths present a problem, outlet orifices may be placed behind a deflecting baffle and some distance below the water surface as shown in Figure 3-5.

Under conditions of continuous treatment at least two settling basins should be built, to allow one to be shut down periodically for cleaning. It is not essential in pretreatment that the basins be designed to handle the full plant capacity at all times. For example, a plant with two presettling basins can allow each basin to carry half the plant flow capacity. When one basin is shut down for cleaning, the other basin can be overloaded for a short period, until both basins are put back into service. When possible, cleaning and sludge removal should be done during periods of low turbidity in the raw water when the shut down of one basin will not overburden the primary treatment units. The process of manual sludge removal is laborious, but normally not necessary more than once a year, depending on basin size. Fire hoses or fixed nozzles can ease the cleaning process (see Chapter 7, "Manual Sludge Removal").

Settling tests, using cylinders 2-5 meters high and 20 cm in diameter; particle-size distribution tests, using a hydrometer analysis; and graphical analyses of the data generated from these tests, are reported in a recent study from Mosul, Iraq involving the design of plain sedimentation basins for several rapid filtration plants that draw water

FIGURE 3-5

Submerged Orifice Basin Outlet System



[SOURCE: Arboleda, 1973, p. 231]

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from the Tigris River (Ahmad, Wais, and Agha, 1982). Four sets of experiments were conducted for water having turbidities of 500 NTU, 1200 NTU, 1800 NTU, and 2500 NTU. The results of the settling tests, presented in Table 3-4, indicates that much of the turbidity was removed within 3 to 4 hours, while a turbidity of 50 NTU was reached in 24 hours in all cases. On the basis of these pilot studies, an overflow rate of 0.5 m/hr, detention time between 2.25 to 4 hours, and basin depth of 2 meters were selected for the design of the plain sedimentation basins. The procedures that were followed in this study may serve as a general guide for the design of plain sedimentation basins, although it should be recognized that the particular results obtained in this study cannot be applied directly to other situations.

Tube or inclined plate settlers can be used for upgrading the pretreatment of water in existing plain sedimentation basins or for reducing the size of new basins (Ahmad and Wais, 180). It was found experimentally that tube settlers were effective in removing turbidity from water containing sand, silt, and clay particles before coagulation. The best tube inclination for turbidity removal, determined by experiment, was 40^{o} from the horizontal. At a surface loading rate of 59 m/day, the turbidity removal was as high as 85% in a 2.6-cm diameter tube. A modular design employing a presettling basin with inclined-plate settlers prior to slow-sand filtration is shown in Figure 3-6 (CEPIS, 1982). The settling unit is operated at a high rate of 60 m/day and has a detention time of 25 minutes. This design is able to treat waters with turbidities up to 500 NTU, but only when the turbidity results from particles larger than 1 micron.

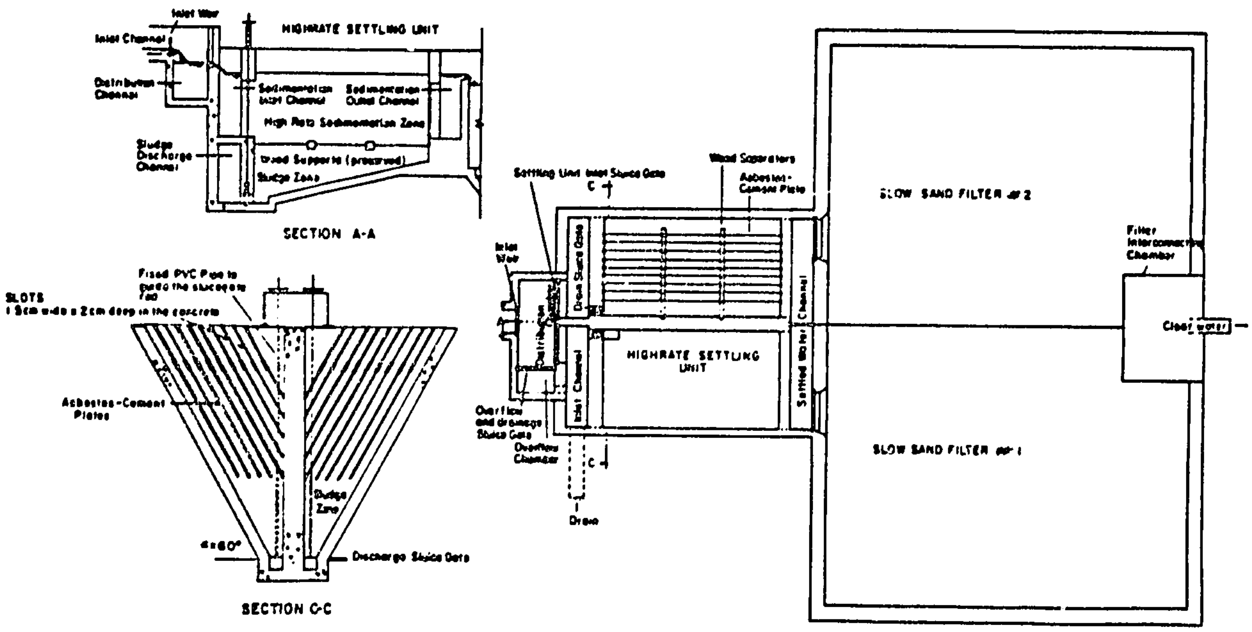
Storage

Storage reservoirs can be used for presedimentation. The detention time is generally much greater than that for conventional sedimentation basins, ranging from about one week to a few months. For extremely turbid rivers or streams (average annual turbidity over 1000 NTU), storage provides the best pretreatment. Storage serves several purposes in water treatment: (1) it reduces the turbidity by natural sedimentation; (2) it attenuates sudden fluctuations in raw water quality; (3) it improves the quality of water by reducing the number of pathogenic bacteria (if the storage site is protected); (4) it improves the reliability of the water supply as it can be drawn upon during periods of short supply of raw water; and (5) it can be drawn upon during short periods of exceedingly high turbidity.

The design of storage basins is not subject to well defined criteria, but should take into consideration local conditions, especially land availability for the

FIGURE 3-6

High-rate Plain Sedimentation with Inclined-plate Settlers
Before Slow-Sand Filtration
(designed by CEPIS for rural communities)



[SOURCE: adapted from CEPIS, 1982, vol. 2, plan nos. 12, 13] construction of the basin. Storage basins may be shaped into ponds or lagoons formed from the natural topography of the earth, or constructed from manmade earthen dams. The capacity of a storage basin should allow for losses due to evaporation and seepage, especially in arid regions. In places where seepage is a problem, the bottom of the basin should be covered with some type of impermeable layer, such as clay or masonry. In some instances, it may be desirable to restrict public access to the storage basin to maintain the quality of the water. A simple method for protecting earthen storage basins is to plant a natural barrier of heavy vegetation, such as thorn bushes, around the periphery of the basin to conceal it and break wind effects as well as to thwart potential polluters.

Sludge removal and basin cleaning for small storage basins are accomplished in a manner similar to those for plain sedimentation. A drain should be provided to remove the water from the basin. After the basin is emptied, it may be cleaned manually by using wheel barrows and shovels to remove the remaining sludge. Because of the large capacity for sludge storage, the cleaning operation can be carried out relatively infrequently.

Smethurst (1979) has compiled extensive plant-operating data on the quality of water before and after storage for several water supply facilities in England that use storage for pretreatment. These data are tabulated in Tables 3-5

TABLE 3-5: Quality of Water Before and After Storage* for Water Supplies in England

	RIVER THAMES AT TEDDINGTON		RIVER THAMES AT OXFORD		RIVER GREAT OUSE AT DIDDINGTON	
	Raw River	Stored Water	Raw River	Stored Water	Raw River	Stored Water
Color, Hazen	830	450	19	9	30	5
Turbidity units (NTU)	35	5.3	14	3.2	10	1.5
Presumptive coli, MPN per 1000 ml	--	--	60,000	200	6500	20
Presumptive coli, % of samples:						
of 100 cm^3	99.9	57.2	--	--	--	--
of 10 cm^3	97.7	32.5	--	--	--	--
of 1 cm^3	83.1	13.4	--	--	--	--
of 0.1 cm^3	48.3	3.3	--	--	--	--
E. coli, MPN per 100 ml	--	--	20,000	100	1700	10
Colony counts per 1 ml.						
3 days at 20°C	4465	208	--	--	50000	580
2 days at 37°C	280	44	--	--	15000	140

*Storage of 7 to 14 days.

[SOURCE: adapted from Smethurst, 1979, p. 22]

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and 3-6. A remarkable improvement in bacteriological quality as well as significant reductions in turbidity due to storage is evident from these tables. Dr. A. V. Houston's studies on bacterial die-away in London's storage basins along the Thames almost a century ago were the basis for water treatment until chlorination came onto the scene.

Roughing Filtration

Particles removed in filters are much smaller than the pore spaces in the media, so the process of filtration is not straining. The principal processes are sedimentation in the pore spaces, adhesion to the media particles and, in slow-sand filters, biochemical degradation of particles that are captured.

Roughing filters allow deep penetration of suspended materials into a filter bed, and have a large silt storage capacity. The solid materials retained by the filters are removed by flushing, or if necessary, by excavating the filter media, washing it, and replacing it. Roughing filtration uses much larger media than either slow or rapid filtration, as indicated in the following comparison:

Slow-Sand Filters	0.15 - 0.35 mm diameter
Rapid-Sand Filters	0.4 - 0.7 mm diameter
Roughing Filters	>2.0 mm diameter

The rates of filtration, however, can be as low as those used for slow-sand filters, or higher than those used for rapid

TABLE 3-6: Change in Water Quality Due to Storage for Water Supplies in England

	REDUCTION DUE TO STORAGE, %	
	River Severn at Hampton Loade	River Derwent at Draycott
Color	28	67
Turbidity	70	51
Presumptive coli	95	99
E. coli	94	>99
Colony counts per 1 ml:		
3 days at 20°C	88	--
2 days at 37°C	89	--

[SOURCE: adapted from Smethurst, 1979, p. 23] filters, depending on the type of filter, and the desired degree of turbidity removal.

Roughing filters are often used ahead of slow-sand filters because of their effectiveness in removing suspended solids. Field studies in Tanzania have shown that, in many cases, neither plain sedimentation nor storage is as effective as roughing filters for pretreating raw water to the physical standards required by slow-sand filters (Wegelin, 1982). Roughing filters are limited, however, to average annual raw water turbidities of 20-150 NTU, so as to prevent too frequent clogging and to ensure their continuous operation for an extended period of time.

There are basically two types of roughing filters which are differentiated by their direction of flow: namely vertical flow (VF), and horizontal flow (HF) roughing filters. Structural constraints limit the depth of the filter bed in VF filters, but higher filtration rates and backwashing of the filter media are possible. On the other hand, HF filters enjoy practically unlimited filter length, but are normally subject to lower filtration rates and they generally require manual cleaning of the filter media.

Vertical Flow Roughing Filters

VF roughing filters are further subdivided into upflow and downflow units. The VF upflow filter can give good results in pretreating raw water with turbidities less than 150 NTU. A typical arrangement for a VF upflow filter is

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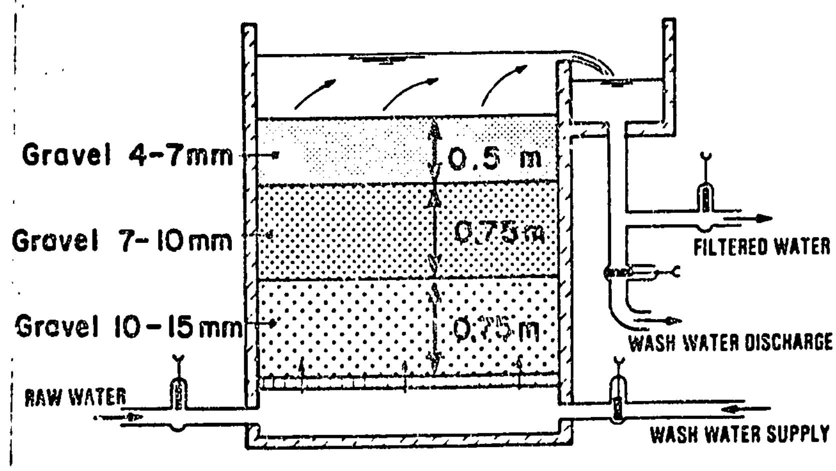
shown in Figure 3-7. Several gravel layers, tapering from a coarse gravel layer (10-15 mm) located directly above the underdrain system, to successively fine gravel layers (7-10 mm and 4-7 mm) effect deep penetration of suspended solids into the filter bed. Filtration rates in gravel upflow filters are relatively high (up to 20 m/hour) because of the large pore spaces in the filter bed which are not likely to clog rapidly. Low backwashing rates are allowable since the bed does not expand; but longer time periods for adequate cleaning of the gravel are usually necessary (about 20-30 minutes). Filter underdrains can be fabricated locally, using either a "teepee" type of design, or a main and lateral system (both are described in Chapter 8, "Filter Bottom and Underdrains"). Upflow filters are used predominantly in upflow-downflow type filtration to replace the unit process of flocculation and sedimentation found in conventional rapid filtration plants (see Chapter 8, "Upflow-Downflow Filtration"). They are similar in design and construction to gravel bed flocculators (see Chapter 6, "Gravel-Bed Flocculators").

VF downflow roughing filters using shredded coconut fibers for the filter medium have been said to be successful in Thailand (Frankel, 1974), and installed in over 100 rural villages in Southeast Asia (Frankel, 1981). The raw coconut husks are found throughout Southeast Asia and have little conventional market value, hence they provide a low-cost

FIGURE 3-7

Gravel Upflow Roughing Filter



[SOURCE: adapted from IRC, 1982b, p. 273]

filter medium for treatment plants in that part of the world.

Shredded coconut fiber may be prepared manually by soaking the husk for 2-3 days in water, and then shredding the husk by pulling off the individual fibers and removing the solid particles which bind the fibers. Shredded coconut fibers may also be purchased directly from upholstery stores or coir (coconut fiber) factories. The shredded fiber should be immersed in water for about three days, until the fiber does not impart any more color to the water (Frankel, 1977). The depth of the coconut fiber in the filter box is usually 60-80 cm. There are no backwashing arrangements for cleaning the coconut fibers as the fibers do not readily relinquish entrapped particles because of their fibrous nature. Instead, water is drained from the filter box and the dirty fibers are removed and discarded. Coconut fiber stock, which has been properly cleaned, is then packed into the filter. The filter media generally must be replaced every three or four months. The availability of the raw coconut husks at low cost, as well as the elimination of backwash pumps and ancillary equipment, combine to make this manual filter bed regeneration process economical in areas where coconut trees are common. The use of such indigenous materials for filter media is also a practical alternative to conventional filter design (see Chapter 8, "Dual-media filters").

Several small filter plants ranging in capacity from 24-360 m^3/day were constructed from 1972 to 1976 in the Lower Mekong River Basin countries (Thailand, Viet Nam, Cambodia) and in the Philippines (Frankel, 1981). Two-stage filtration, using shredded coconut fibers and burnt rice husks for the roughing and polishing filters respectively, was typical for all filter plants. The filtration systems generally produced a clear effluent (less than 5 NTU) when treating raw water with a turbidity less than 150 NTU. The units were designed at a filtration rate of 1.25-1.5 m^3/hr, which is about 10 times higher than that used for conventional slow sand filters. Bacterial removals averaged 60-90% without the use of any disinfectant. The media generally required changing once every 3-5 months depending on the level of turbidity in the raw water.

Horizontal Flow Roughing Filters

Horizontal flow (HF) filters have a large silt storage capacity because of their coarse filter media and long filter length. Filter operation commonly extends over a period of years before the filter must be removed from service and cleaned. HF pretreatment filters have been operating successfully ahead of slow sand filtration at several water treatment plants in Europe (Kuntschik, 1976). On the basis of several pilot projects conducted on HF filters which have substantiated their effectiveness in pretreating turbid river waters in Thailand, Tanzania, and

Honduras, their use in developing countries holds much promise (Thanh, 1978; Wegelin, 1982; CEPIS, 1982).

The main features of a HF roughing filter are shown in Figure 3-8. For overall efficiency, it is best to use a graded gravel scheme for the filter medium. The HF filter is usually divided into several fraction zones, each with its own uniform grain size, tapering from large sizes in the initial zone to small sizes in the final zone. In this way, penetration of suspended solids will more easily take place over the entire filter bed and result in longer filter runs. The following design guidelines have been suggested by Wegelin (1982) and are based on extensive field testing of HF roughing filters ahead of slow sand filters in Tanzania:

1) The acceptable range for the filtration rate is 0.5-4.0 m/h, but an upper limit of 2.0 m/h should be observed for waters with very high suspended solids load and/or colloidals.

2) The filter grains to be used should have two to three fraction zones with sizes ranging from 4-40 mm. The sequence of arrangement in the longitudinal direction should be from coarse to fine.

3) Because the first fraction zone of the filter bed stores a higher percentage of suspended solids than the others, the length of the coarse zone provided should be greater than that of the finer zones in order to provide a large silt storage volume. Thus, the following range of lengths of individual fraction zones should be provided:

   first, coarse fraction: 4.5 - 6.0 m
   middle, medium fraction: 3.0 - 4.0 m
   last, fine fraction: 1.5 - 2.0 m

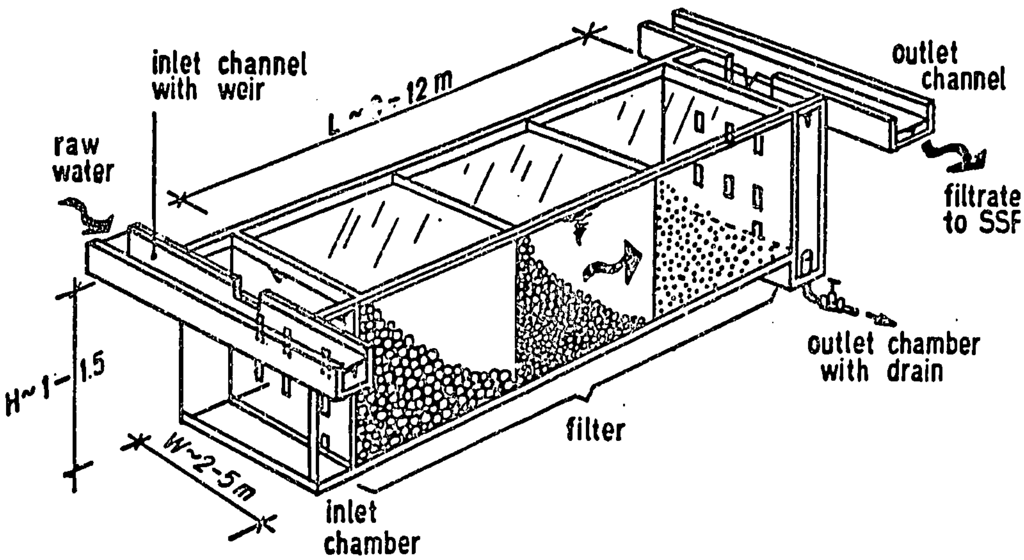
   As a result, the total length of filter should be 10.0 - 12.0 m.

4) For HRF with side walls which are above the ground surface, the height should be below 1.0 - 1.5 m to allow for easy cleaning of the HRF which will involve manual digging out of gravel and refilling it after cleaning.

5) The free water table in the HRF should be covered by a 10-20 cm thick gravel layer in order to prevent plant and algal growth. Hence, the top level of the filter medium should be 30-40 cm above the crest level of the outlet weir.

FIGURE 3-8

Basic Features of a Horizontal-Flow Roughing Filter



[SOURCE: Wegelin, 1982]

6) The filter floor should slope in the direction of flow (about 1:100) so that the filtered water can maintain sufficient velocity.

7) The outlet weir should be provided with a V-notch weir to facilitate discharge measurements.

The filter length is the most critical dimension in the design of HF filters and should be selected after considering an appropriate balance between construction costs and the frequent cleanings required when filter lengths are short. Consideration of such a balance led to the construction of several HF gravel filters in the City of Dortmund, West Germany, each of a length of about 50-70 meters to pretreat raw water from the Ruhr River (Kuntschik, 1976). The total operating period for these extremely long filters is about 5 years; after which the gravel has to be removed, cleaned, and replaced. The high cost of labor in countries like West Germany dictates the design of long filters to minimize the frequency of cleaning, which is relatively expensive. For developing countries, however, such great lengths are not usually warranted; instead filter lengths between 4 and 15 meters seem reasonable because the cleaning of the filter can be accomplished at much lower cost. Of more concern for design purposes, however, is the availability of the filter media. The gravel or crushed stone that is required

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for HF prefilters must be of reasonably uniform size, which may be difficult to obtain in large enough quantities if large filter beds have to be filled.

An HF prefilter has been operating successfully prior to slow-sand filtration for the village of Jedee-Thong, Thailand (Thanh, 1978). The design incorporates 6 gravel zones in a filter box with a volume of 6x2x1 meters (Figure 3-9). The characteristics of the filter media are as follows:

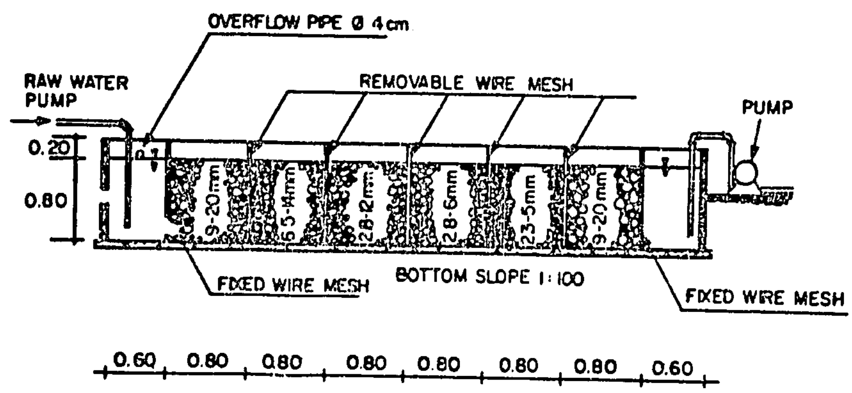
SIZE RANGE mm	EFFECTIVE SIZE (P10), mm	UNIFORMITY COEFFICIENT (P60/P10)
9 - 20	15	1.38
6.5 - 14	11	1.5
2.8 - 12	6.1	1.47
2.8 - 6	3.8	1.36
2.3 - 5	2.6	1.27
9 - 20	15	1.38

The filter box was constructed from bricks covered with a layer of fine mortar. The 6 compartments are separated by removable strong wire mesh which allows for easy cleaning and changing of the media. The filtering area is preceded and followed by chambers without gravel, the effluent chamber serving as a wet well for the pumps (see Figure 3-9). Thanh (1978) reported a removal efficiency of 60-70% for this filter for raw water turbidities ranging from 30 to 100 NTU.

For larger communities, the horizontal-roughing filter depicted in Figure 3-10 may be appropriate. The filter is designed to be constructed adjacent to a stream bed so as to allow raw water to flow through a porous stone wall and into a gravel bed. The drain system is made of a perforated PVC pipe which leads to a junction box. To avoid the infiltration of surface runoff, an impermeable layer of clay or a polyethylene liner can be placed over the gravel bed. This design is intended to operate at a filtration rate of

FIGURE 3- 9

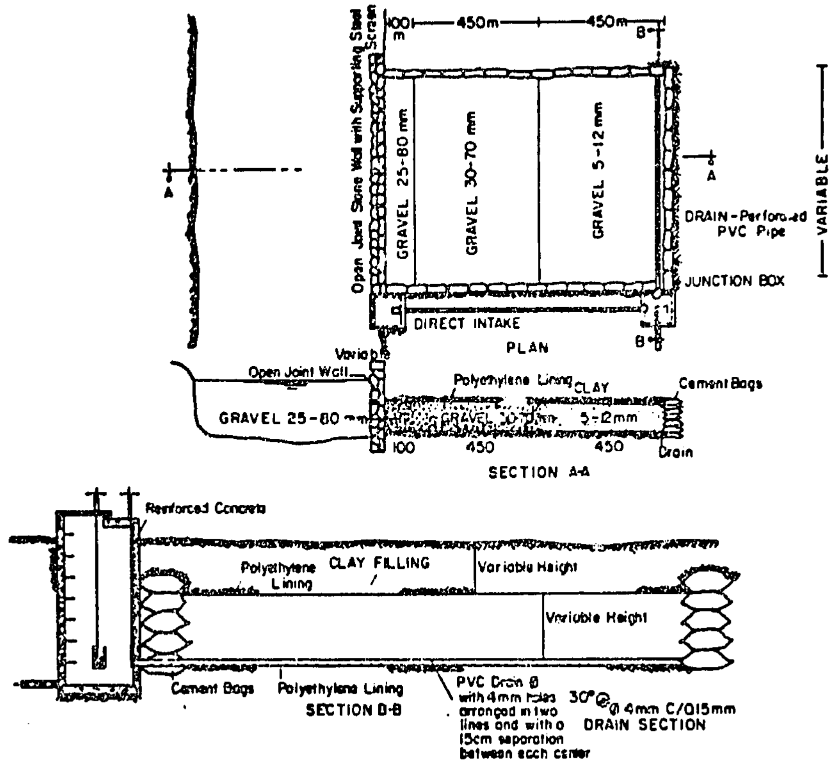
Horizontal-flow Roughing Filter Used Before
Slow-Sand Filtration in Jedee-Thong, Thailand
(dimensions are in meters)



[SOURCE: Thanh, 1978, p. 17]

FIGURE 3-10

Horizontal-flow Roughing Filter Constructed Adjacent
to a Stream Bed
(designed by CEPIS for rural communities)



[SOURCE: CEPIS, 1982, vol. 2, no. 3]

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0.5 m/hr., and can treat waters of turbidities less than 150 NTU prior to slow sand filtration. The length of the filter is variable, depending on the design capacity. The state agency SANAA, in Honduras, is currently field testing such roughing filters as part of their rural water supply program (CEPIS, 1982).

Chemical Pretreatment

Chemical pretreatment is generally not needed for turbidity removal prior to rapid filtration plants, as coagulants are always added at the rapid mix chamber. It is helpful, however, for (1) reducing seasonal influxes of high turbidity in raw water that is treated by slow-sand filtration; and (2) controlling the growth of algae in storage reservoirs.

In places where slow-sand filters are used for water treatment, seasonal peaks in raw water turbidity must be attenuated so that the recommended limit of 50 NTU is not surpassed for extended periods of time. In such cases, chemical pretreatment in presettling basins may be beneficial, particularly if the suspended matter in the water is colloidal. Natural coagulants may be economically attractive chemicals to use for pretreatment purposes (see Chapter 4, "Natural Polyelectrolytes").

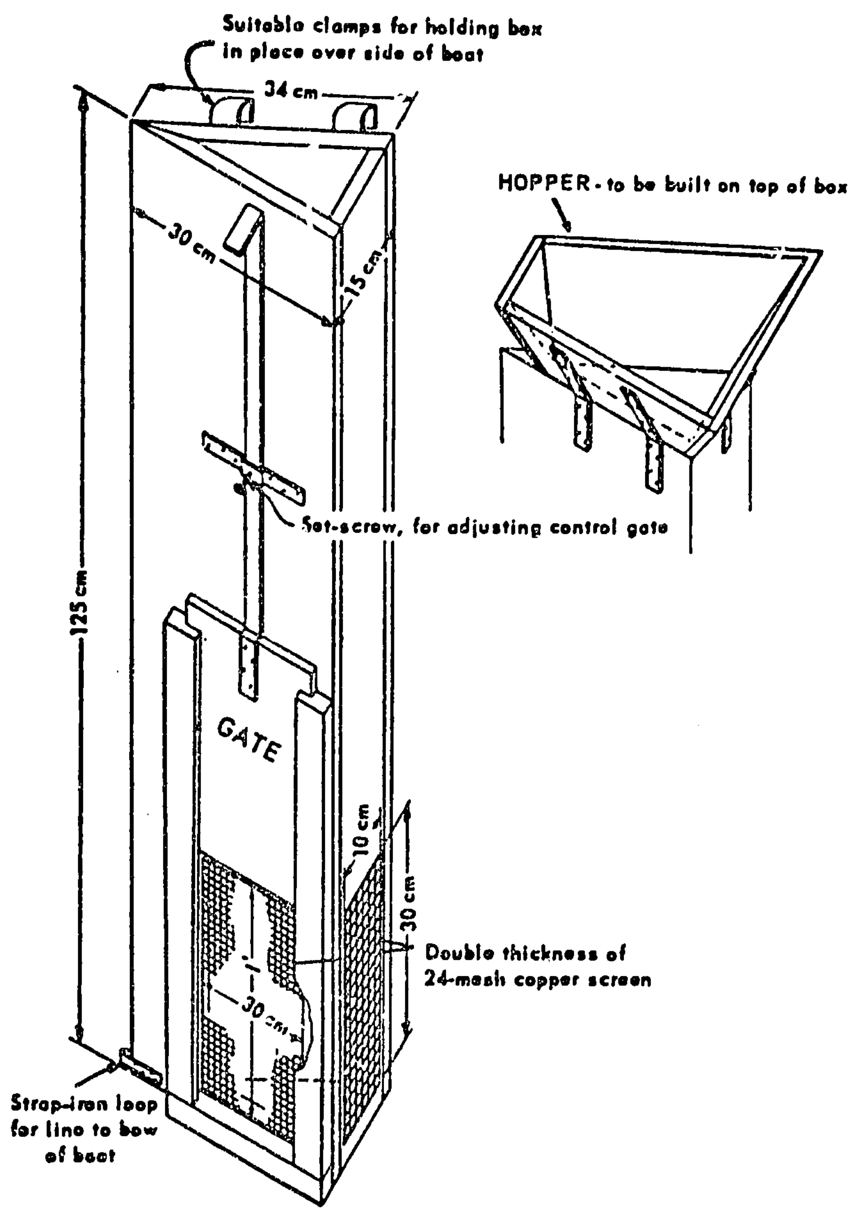
Storage of water in an open reservoir provides an opportunity for algae to grow and develop. The greater the concentration of nutrients in the water, the larger will be the growth of algae. The potential for algal growth in a particular area can be found by observing ponds or lakes in that area. In the industrialized countries, microstrainers are often used for the removal of algae and other plankton organisms from drinking water supplies. Such screening mechanisms are foreign exchange items and they clog easily, and therefore must be frequently cleaned by automatic high-pressure sprayers or mechanical raking devices. A better method for developing countries is to use an algicide such as copper sulphate, if available at reasonable cost. When algicide treatment is deemed appropriate, it is desirable to apply it at the first signs of algal growth, so as to reduce the amount of decaying matter which can cause taste and odor problems in drinking water. The required dose is a function of the types of organisms and their relative numbers and, therefore, microscopic examination of samples of the impounded water is desirable. In the absence of laboratory testing, a dose of 0.3 mg/l is usually sufficient when copper sulphate is used, except when alkalinity is high (Cox, 1964). When alkalinity is high, Cu is precipitated as $CuCO_3$.

In the treatment of reservoirs, copper sulphate may be applied by two simple methods; viz. the burlap bag method and the wooden box method. The first method is accomplished by hanging burlap bags containing copper sulfate crystals from the sides or stern of small rowboats. The boat is usually propelled in parallel courses about 8 to 15 meters apart. Boats equipped with outboard motors greatly improve the mixing of the chemicals in the water due to the action of the propeller. The second method is simply an improvement on the first method, whereby the burlap bags are replaced by permanent boxes attached to both sides of a small boat. The design for a copper sulphate distribution box is shown in Figure 3-11. The rate of solution of the crystals can be controlled by changing the position of the control gate. Cox (1964) gives further information on the use of copper sulphate in water treatment including dosage requirements, toxicity, and frequency and method of application.

Chlorine and its compounds may also be used to suppress algal growths. They are more effective, however, in small reservoirs so that a residual chlorine concentration can be maintained at a reasonable cost. Doses of chlorine up to 1 mg/l and higher, which produce residuals up to 0.50 mg/l have been found effective with many organisms.

FIGURE 3- 11

Box for Controlled Distribution of Copper Sulfate
Solution in Lakes or Reservoirs



[SOURCE: Cox, 1964, p. 42]

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IV. CHEMICALS AND CHEMICAL FEEDING

The chemicals necessary in water treatment plants include a coagulant, generally alum; disinfectants, generally chlorine; and, when necessary, alkalies for pH control, generally lime. Coagulant aids may also be used to improve the coagulation-flocculation process, or to reduce coagulant consumption. Fortunately, alum, chlorine, and lime are the most readily available water treatment chemicals in developing countries, albeit expensive when imported. Other types of water treatment chemicals widely used in the industrialized countries to provide fluoridation, taste and odor removal, and stability and corrosion control are not recommended for communities in developing countries, except perhaps in the major cities where skilled supervision and the chemicals are available.

The improper selection, handling, and feeding of chemicals can be detrimental to water treatment plant performance, and have been the bane of many such plants in developing countries. A survey of plants conducted in India (NEERI, 1971) revealed that about 80% of the plants were dosing alum in an unscientific and primitive way (by dumping blocks of alum into the raw water channels), since alum equipment was out of order. Similarly, in 50% of the plants studied, the chlorine dosing equipment was out of order and chlorination was done by bubbling chlorine gas directly into the filtered

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water channel. Bacteriologically safe water was not being produced in most cases, and no plant had the capacity to switch over to break-point or superchlorination under emergency conditions. To avoid problems in small communities in developing countries, alternative chemicals that are easily handled and applied should be explored, for example, hypochlorite compounds in place of chlorine gas. Similarly, chemical feeders should be simple in design and easy to operate. Whenever possible, the local manufacture of these items is to be preferred over their importation.

This chapter begins with a brief discussion of the jar test, which is the standard laboratory procedure for selecting chemicals and optimal doses. This is followed by sections on the primary coagulants, alkalies, natural coagulant aids, disinfection, and chemical feeders. The Table of Chemicals Used in Water Treatment, found in Appendix A, summarizes the characteristics of different chemicals.

The Jar Test

The required chemical dosage for a particular raw water is virtually impossible to determine analytically because of the complex interrelationships which exist between these chemicals and the constituents of the water being treated, as well as such factors as pH, temperature, and the intensity and duration of mixing. Consequently, a laboratory procedure known as the "jar test" is used to determine the most effective and economical dose of coagulant for a particular mixing intensity and duration. A brief description of the jar test is presented here (IRC, 1981b).

> A series of samples of water are placed on a special multiple stirrer and the samples are dosed with a range of coagulant, e.g. 10, 20, 30, 40, and 50 mg/l; they are stirred vigorously for about one minute. Then follows a gentle stirring (10 minutes) after which the samples are allowed to stand and settle for 30 to 60 minutes. The samples are then examined for color and turbidity and the lowest dose of coagulant which gives satisfactory clarification of the water is noted.
>
> A second test involves the preparation of samples with the pH adjusted so that the samples cover a range (e.g. pH = 5, 6, 7, and 8). The coagulant dose determined previously is added to each beaker. Then follows stirring, flocculation, and settlement as before. After this, the samples are examined and the optimum pH is determined. If necessary, a re-check of the minimum coagulant dose at the optimum pH can be done.

The times for stirring and settlement may be reduced based upon experience without affecting the results. Laboratory stirring equipment, such as the one shown in Figure 4-1, provides uniform mixing for a number of samples simultaneously and can be adjusted to match plant scale velocity gradients for rapid mixing and flocculation (formulae for calculating plant scale velocity gradients are given in Chapter 5, "Design Criteria"). These units may be purchased from laboratory supply houses or manufactured

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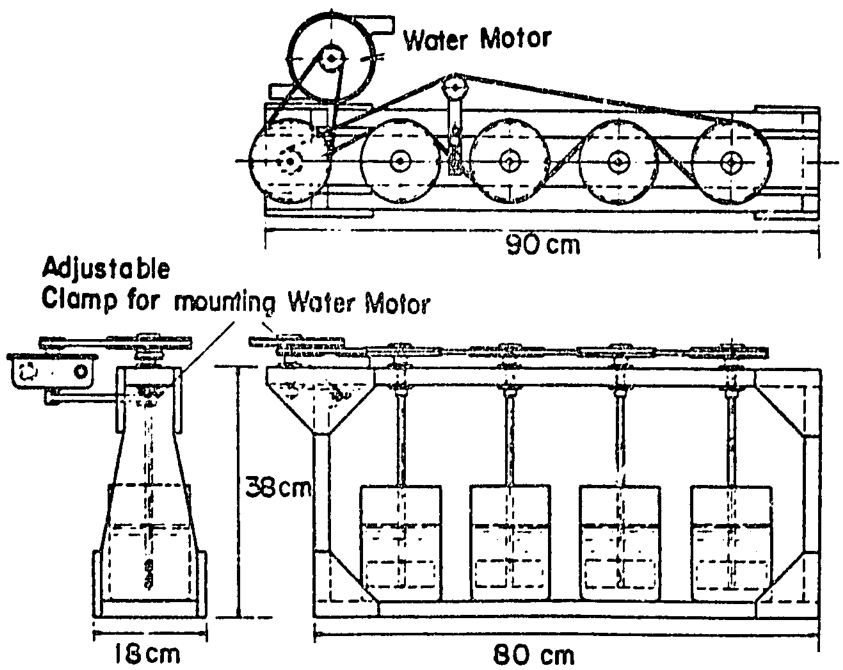
FIGURE 4-1

Laboratory Stirring Equipment for Coagulation and Flocculation or Jar Test



[SOURCE: Cox, 1964, p. 314]

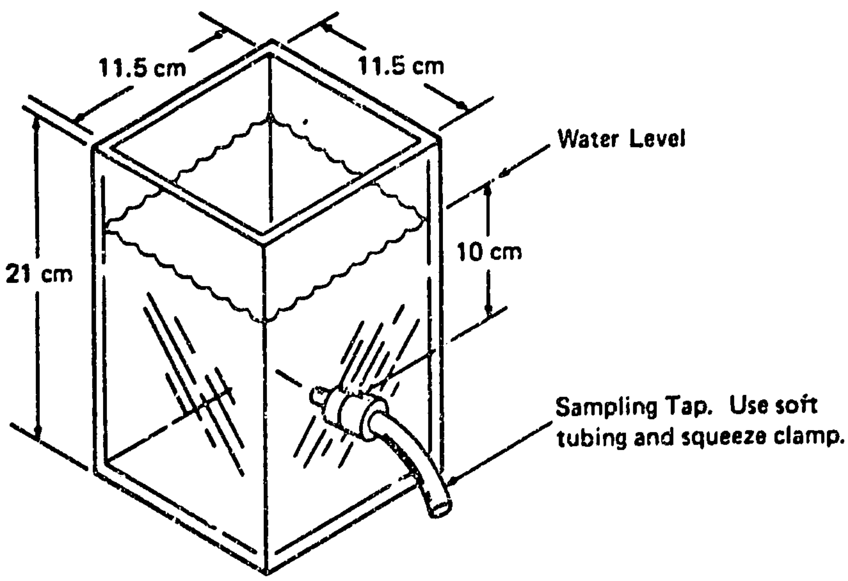
locally. The multiple stirrer shown in Figure 4-1 is powered by a water motor, but units operated by a hand crank or a small electric motor with speed reducing systems may also be used. Metal rods with stirring paddles are attached to pulleys suspended directly over the beakers, which rotate at the same rate due to the common drive band. The rods have a handle attached to the upper end so that they may be lifted vertically while the pulleys are turning.

Standard 2-liter laboratory beakers are commonly used for jar testing; but an effective alternative uses square plastic jars (Hudson, 1981), which inhibits vortex formations at high stirring velocities. The 2-liter square jar, shown in Figure 4-2, has a side outlet tap which has been found to be more convenient to use than sampling siphons or pipettes. The calibration curve presented in Figure 4-3 was developed for rectangular jars and plots velocity gradient (G) versus agitation rpm at four different temperatures.

A complete discussion of jar testing and the utilization of jar test data are given by Hudson (1981). A methodology is outlined that establishes standardized, fixed procedures for conducting and evaluating jar tests. Other authors have also addressed the subject (Cox, 1964; Hardenbergh and Rodie, 1961; AWWA, 1971; Fair, Geyer, and Okun, 1971).

FIGURE 4-2

Two-liter Jar for Bench-scale Testing.
Fabricate from Plexiglass Sheet or Similar Material.



[SOURCE: Hudson, 1981, p. 47]

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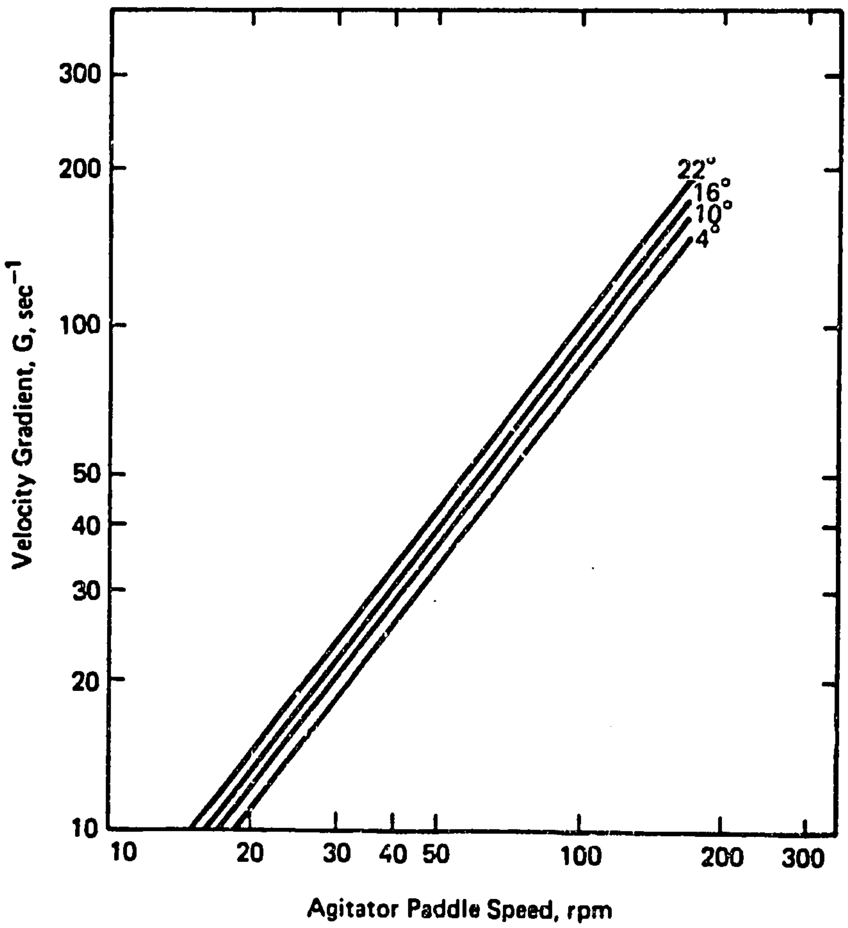
FIGURE 4-3

Velocity Gradient vs RPM for a Two Liter Square Beaker, using a stirrer with a 3 in x 1 in paddle held 2¼ in. above the bottom of the beaker



[SOURCE: Hudson, 1981, p. 47)

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Primary Coagulants

Metal coagulants based on aluminum, and to a lesser extent, based on iron, are used almost exclusively in the coagulation process. The choice of coagulant should be determined under laboratory conditions, e.g. jar testing, with the final choice influenced by economic considerations.

Alum Salts

Aluminum (aluminum sulfate; $Al_2(SO_4)_3.14H_2O$) is available commercially in lump, ground, or liquid form. Dry alum (density about 480 kg/m^3) is measured by volume or weight and is normally dissolved in water prior to its introduction into the rapid mixer. The content of water-soluble alumina in dry alum is 11-17%. Figure 4-4 shows 100-kg bags of lumped alum manufactured in Kenya. This form of alum is easily handled and stored in the treatment plant. Low-grade lumped alum has been used effectively in saturation towers in Latin America (see "Saturation Towers" below). Liquid alum may be obtained economically if alum-producing industries are nearby (e.g. to serve large paper mills). It is usually cheaper than dry alum when obtained at the source, but the shipping weight is double or more(e.g. 1270 kg/m^3 for 7.2% Al_2O_3-content grade) and requires special shipping containers because of its corrosiveness. The water soluble alumina content in liquid alum is 5.8 to 8.5%.

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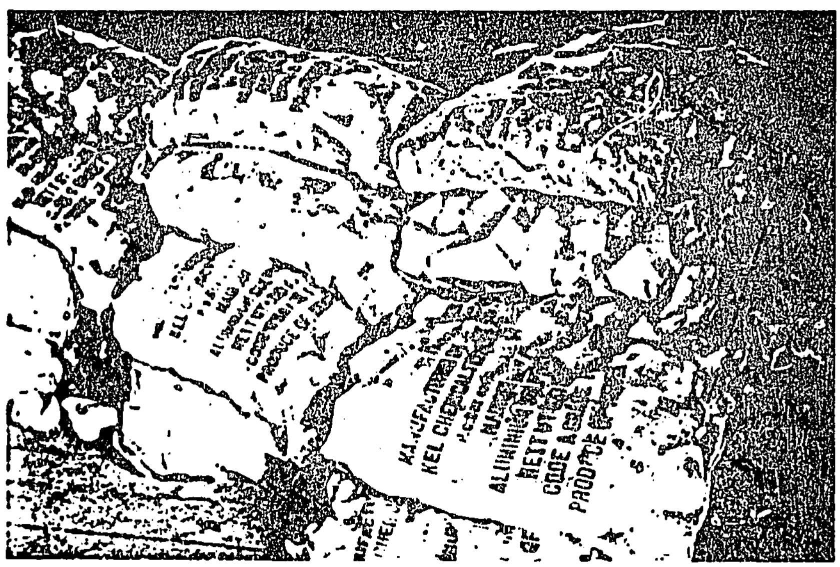
FIGURE 4-4

100-kg Bags of Lumped Alum Stored at a Treatment Plant
in Nairobi, Kenya



[SOURCE: Singer, personal communication]

Alum is a relatively inexpensive coagulant if local production is possible. However, in some developing countries, alum must be imported at substantially increased costs. Countries in West Africa, for example, import most of their alum from Europe, paying as much as $700/ton (Wagner and Hudson, 1982). This compares with a price of about $100/ton for commercially produced alum in the United States. Accordingly, treatment plants in those areas should be designed so that alum consumption is minimized. The dosage of alum may be reduced in some instances by (1) pretreating excessively turbid river waters; (2) direct filtration of low turbidity (<50 NTU) waters; or (3) the use of coagulant aids.

The correct alum dosage is determined initially from jar tests of the raw water, and then modified by actual plant operation experience. Optimal floc formation using alum occurs when the pH value of the water is between 6.0 and 8.0. If insufficient alkalinity is present to react with the alum, an alkali such as lime must be added. The reactions of alum with the alkalinity in the water are impossible to determine accurately, but the following quantities serve as a useful guide (AWWA, 1971):

1 mg/l of alum reacts with-----

0.50 mg/l natural alkalinity, expressed as $CaCO_3$

0.33 mg/l 85% quicklime as CaO

0.39 mg/l 95% hydrated lime as $Ca(OH)_2$

0.54 mg/l soda ash as Na_2CO_3

These approximate amounts of alkali, when added to water, will maintain the water's alkalinity when 1 mg/l of alum is added. For example, if 1 mg/l of alum is added to raw water, the alkalinity will drop by 0.50 mg/l; however, if 0.39 mg/l of hydrated lime is added with 1 mg/l of alum, the alkalinity of the raw water will remain the same.

A suitable method for feeding alum in developing countries is via solution-type chemical feeders. The alum is dissolved in water at a concentration of 3% to 7% (5% is most commonly used) in tanks, and then fed to the raw water. The highest concentration that can be practically achieved in alum solutions is 12 to 15% by weight. Such saturated solutions are used in alum saturation towers.

Ferric Salts

Four types of ferric salts are used as coagulants: (1) ferrous sulfate (copperas); (2) chlorinated copperas; (3) ferric sulfate; and (4) ferric chloride. The physical and chemical characteristics of each are summarized in Appendix A and covered more fully in standard references (AWWA, 1971; Cox, 1964). In general, they give similar results when their doses are compared in terms of iron content.

A number of practical differences between alum and ferric coagulants have been noted by Cox (1964):

1) Ferric hydroxide is insoluble over a wider range of pH values than aluminum hydroxide. This is illustrated

in the pH zone-coagulation relationship shown in Figure 4-5 for aluminum sulfate and ferric sulfate. The two curves clearly indicate that for alum the pH zone for optimal coagulation is relatively narrow (6.5 to 7.5), whereas for ferric sulfate it is much broader, ranging from 5.5 to 9.0.

2) Ferric hydroxide is formed at low pH values, so that coagulation is possible with ferric sulfate at pH values as low as 4.0 and with ferric chloride at pH values as low as 5.0.

3) The floc formed with ferric coagulants is heavier than alum floc;

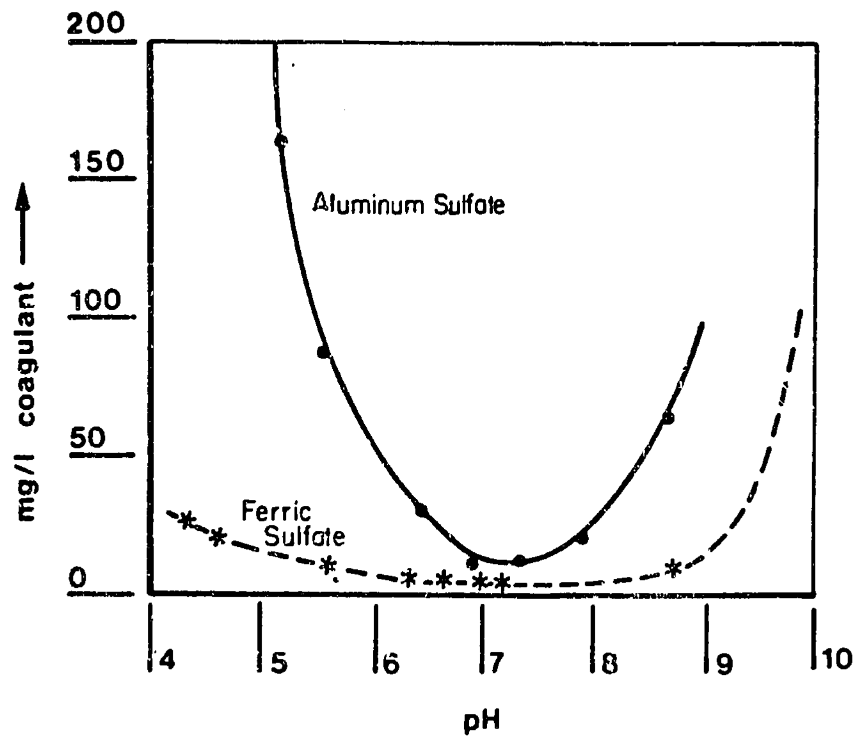
4) The ferric hydroxide floc does not redissolve at high pH values;

5) Ferric coagulants may be used in color removal at the high pH values required for the removal of iron and manganese and in the softening of water.

Iron coagulants are now recovered from steel mill waste pickling liquor and have become increasingly competitive in recent years, often yielding superior results at lower cost than alum. Although ferric salts are not as widely available as alum, which may prevent their widespread adoption as coagulants in developing countries, the possibility of using ferric salts should be investigated.

FIGURE 4-5

pH Zone-Coagulation Relationship*



*Coagulation of 50 mg/l kaolin with aluminum sulfate and ferric sulfate. Comparison of pH zones of coagulation of clay turbidity by aluminum sulfate, and ferric sulfate. Points on the curves represent coagulant dosage required to reduce clay turbidity to one-half its original value.

[SOURCE: AWWA, 1971, p. 88]

<u>Alkalies for pH Control</u>

If the natural alkalinity of the raw water is insufficient to react with the dosage of coagulant that is added, i.e. the pH drops below the range for optimum coagulation, then alkalies should be added. Three types of alkalies are suitable for use in developing countries: 1) soda ash (sodium carbonate, Na_2CO_3); 2) quicklime (calcium oxide, CaO); and 3) hydrated lime (calcium hydroxide, $Ca(OH)_2$). Caustic soda (NaOH), a chemical often used in the industrialized countries, is not generally recommended because of its cost and its highly corrosive nature and the extreme care that must be exercised in its handling.

Soda ash is a white powder that is easily soluble in water so that there is less difficulty in its feeding than with other alkalies. Nevertheless, it can cake in storage bins because of its hygroscopic nature. Solutions of soda ash will not clog dosing orifices or feeding lines and, unlike lime, do not have to be stirred continuously. Furthermore, soda ash provides CO_3 ions to aid in corrosion control. The cost of soda ash is normally about three times that of lime, but in countries that mine soda ash, it provides a relatively trouble-free and economic means for alkalinity control.

Lime is produced in either unslaked (quicklime) or slaked (hydrated) form. Quicklime must be slaked (i.e. hydrated, by using a small amount of water) before it can be used. Hydrated lime, however, does not have to be slaked, does not deteriorate when stored, and contains fewer impurities than most quicklimes, making the clogging of orifices and pipelines less of a problem. For these reasons, hydrated lime is preferred when it is available economically. Quicklime is sometimes used at softening plants and large filtration plants because of its lower cost.

Lime is relatively insoluble in water and in most instances is fed as a suspension. Solution tanks and feed lines that store and convey lime-water suspensions clog easily, and must be routinely cleaned. Provision for easy cleaning is essential in the design. To minimize feeding problems, the dilution water should be cold, as lime is more soluble in cold water than in warm water. An alternative to feeding lime-water suspensions is the use of lime saturation towers which yield saturated solutions of lime, and have reduced maintenance problems in Brazil (Arboleda, 1973). Lime saturators are discussed in some detail below (see section on "Saturation Towers").

Natural Coagulant Aids

A great variety of both natural and synthetic materials is available to aid in the clarification of water. The correct application of these coagulant aids may improve the settling characteristics and toughness of the floc which in turn permit shorter sedimentation periods and higher rates of filtration. More importantly, though, such aids may significantly reduce the required dosage of the primary coagulant (e.g. alum), which is beneficial to those developing countries that must import coagulants.

A number of synthetic chemicals (e.g. cationic, nonionic, and anionic polyelectrolytes) have been developed by chemical manufacturers in the United States and Europe that can successfully cope with certain types of coagulation-flocculation problems; especially those arising from seasonal changes in water quality and ambient temperature. In general, however, the use of these chemicals in developing countries is inappropriate, due to the need for their importation, careful monitoring and regulation, and high cost. Continued supply may be questionable. A reasonable alternative, then, is natural coagulant aids, which are available at low cost in most developing countries.

Natural coagulant aids are classified as (1) adsorbents-weighting agents; (2) activated silica; and (3) natural polyelectrolytes.

Adsorbents-Weighting Agents

Bentonitic clays, fullers earth, and other adsorptive clays are used to assist in the coagulation of waters containing high color or low turbidity. They supply additional suspended matter to the water upon which flocs can form. These floc particles are then able to settle rapidly due to the high specific gravity of the clay with respect to the water. Some clays swell when added to water and can produce a floc when used alone or with a limited dosage of alum. Practical experience has shown that doses of clay ranging from about 10 to 50 mg/l result in good floc formation, improved removal of color and organic matter, and a broadening of the pH range for effective coagulation (AWWA, 1971). For low turbidity raw waters (less than 10 NTU), the addition of adsorptive clays may often reduce the dosage of alum required. For example, a dose of 10 mg/l bentonite clay and 10 mg/l alum may give better results than the optimal dose for alum alone (Cox, 1964).

Powdered calcium carbonate (limestone) is also effective as a weighting agent, and, in addition, supplies alkalinity to the water upon dissolving. It is a common construction material (known as whiting in the building industry) and is easily stored, handled, and applied. A dosage of about 20 mg/l is sometimes used to treat low turbidity waters (Cox, 1964).

Activated Silica

Prior to the development of synthetic polyelectrolytes, activated silica was the most widely used coagulant aid in water treatment. It is manufactured by partially neutralizing sodium silicate with an acid reagent such as sulfuric acid or chlorine solution, under carefully controlled conditions so as to prevent the formation of silica gel, which can clog tanks and feeder lines. The preparation and feeding of activated silica is outlined by Walker (1955). Packham and Saunders (1966) describe its physical and chemical properties and its effectiveness as a coagulant aid.

In Kenya, regarded generally as the most developed country in East Africa, and which has the technical capability to utilize more advanced treatment methods, activated silica has been used in several urban plants to improve coagulation at relatively low cost. Nevertheless, in most small communities in developing countries, the preparation of activated silica is too complicated; instead, preference should be given to those types of natural coagulants that need only to be dissolved and proportioned into the water.

Natural Polyelectrolytes

Polyelectrolytes are either derived from natural sources or synthesized by chemical manufacturers. In both instances, their structure consists of repeating units of small molecular weight, chemically combined to form a large molecule of colloidal size, each carrying electrical charges or ionizable groups. Polyelectrolytes are often classified by the type of charge they carry. Thus polymers possessing negative charges are "anionic", those possessing positive charges are "cationic", and those that carry no charges are "nonionic". A wide variety of nonionic polymers is derived from natural sources.

The application of synthetic polyelectrolytes as coagulant aids in water treatment is only appropriate in the industrialized countries, or in countries like Brazil, Argentina, Colombia, and India which have reasonably developed water supply infrastructures that are able to regulate and monitor the manufacture and dosage of those chemicals. For example, a polyacrylamide gel, manufactured in India under the trade name "Polymix", has been used in conjunction with alum in the Delhi Water Works for about one year. The cost of the Polymix Gel is presently Rs. 10 (US$1.10) per Kg. It was found that the maximum dose required did not exceed 0.3 mg/l of Polymix Gel and 10 mg/l of alum, even during the rainy season when turbidity levels are at their highest. An overall savings of 30% in the cost of chemicals used for coagulation was achieved (Singh, 1980).

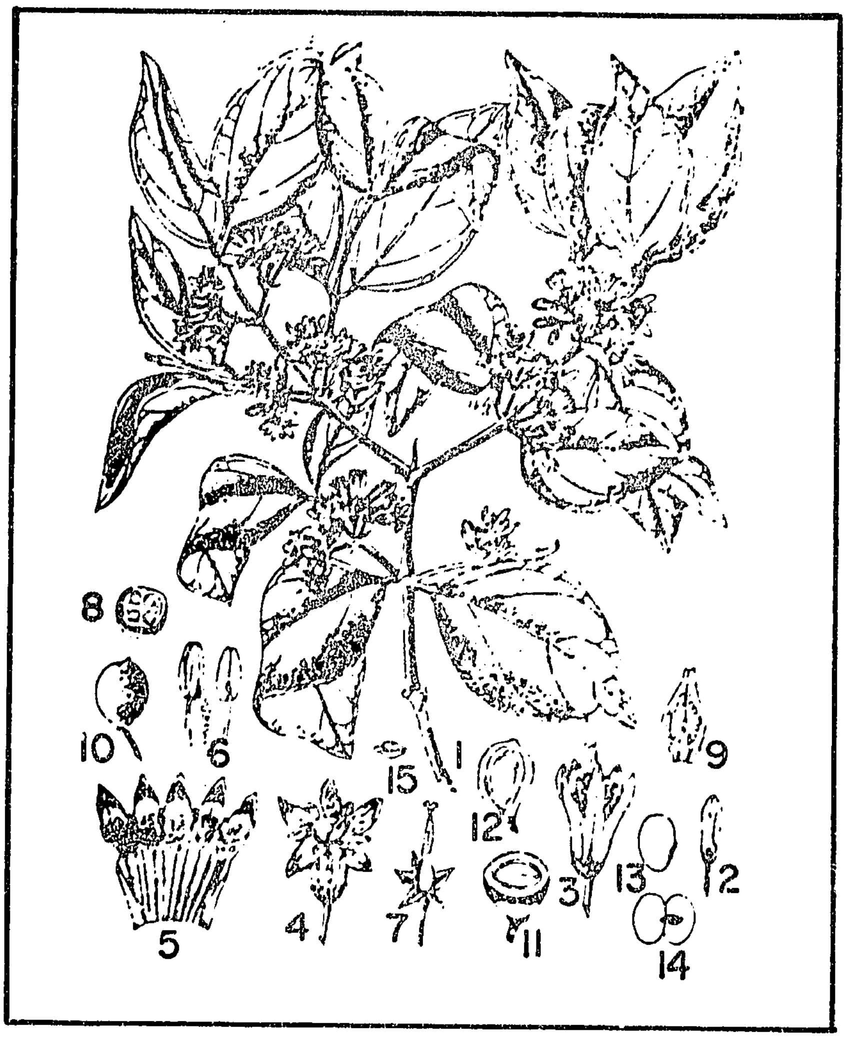
A report published by the IRC (1973) summarizes the health aspects of using synthetic polyelectrolytes in water treatment and outlines procedures for their control that have been adopted in the United States and England and have been effective. Nevertheless, in most developing countries, natural coagulant aids are greatly to be preferred, as they do not require such regulatory control and are usually less costly.

Interestingly, natural polyelectrolytes have been used in developing countries for clarifying water for many centuries. Sanskrit writings from India reported that seeds of the nirmali tree (Strychnos potatorum), illustrated in Figure 4-6, were used to clarify turbid river water 4,000 years ago. In Peru, water has been traditionally clarified with the mucilaginous sap of "tuna" leaves obtained from certain species of cacti (Kirchmer, Arboleda, and Castro, 1975). Jahn (1979) reports that in several countries in Africa (Chad, Nigeria, Sudan, and Tunisia) indigenous plants are added to drinking water by rural villagers to remove turbidity or unpleasant tastes and odors. Thus, the clarifying powers of natural polyelectrolytes are known to the rural inhabitants of numerous developing countries. At the same time, though, these substances also have been proven effective as coagulant aids in community water treatment, based on practical experience with such aids in Great Britain, and research undertaken in several developing countries.

The British were among the first to use natural polyelectrolytes as coagulant aids in urban water supplies (Manual of British Water Engineering Practice, 1969; Packham, 1967). Sodium alginate, a natural polymer extracted from brown seaweed, has been employed by a number of water authorities at doses of 0.4 to 0.5 mg/l as an aid to alum, particularly during periods of low temperature.

FIGURE 4-6

Fruit of Strychnos Potatorum (nirmali seeds)*



*The fruit (10) is a shiny black berry the size of a cherry. Its two stones (11-14) are the "clearing nuts" that are effective as coagulants.

[SOURCE: Jahn, 1981, p. 21]

Sodium alginates are widely used as thickening and stabilizing agents in the food, textile printing, and paper industries. Other natural polymers that have been successfully used in England are hydroxyethyl cellulose (HEC) and Wisprofloc, a derivative of potato starch. Starch products, cellulose derivatives, and alginates are all used in food processing.

The National Environmental Engineering Research Institute (NEERI) in India has completed studies on several plant species to determine their effectiveness as coagulant aids (NEERI, 1976; Tripathi et al., 1976). Seeds from the following plants were studied:

1) nirmali tree; _Strychonos potatorum_
2) tamarind tree, _Tamerindus indica_
3) guar plant; _Cyamopsis psoraloides_
4) red sorella plant; _Hibiscus sabdariffa_
5) fenugreek; _Triconella foenum_
6) lentils; _Lens esculenta_

Laboratory, pilot plant, and full scale plant studies were conducted using raw water turbidities that ranged from 50 to 7500 NTU. The following conclusions were reached:

1. The effective dose is 2 to 20 mg/l in the pH range 4 to 9.

2. The aids are uneconomical for water of turbidity below 300 NTU.

qv

3. The aids are effective at high turbidity levels, during which 40-54 percent savings in alum consumption are possible.

4. The aids deteriorate in about three months even after the addition of preservatives.

An independent study by Bulusu and Sharma (1965) verified the effectiveness of nirmali seeds as a coagulant aid to alum. In a pilot plant, water from the Yamuna River in India with a raw water turbidity of 1200 to 1530 NTU was treated with 10 mg/l alum and 1-5 mg/l crushed nirmali seeds. The resulting settled water turbidity was 22-29 NTU, while the alum consumption was reduced by 74 to 78%.

Trials with coagulant aid treatment in the water works operation of the Kanhan Plant, Nagpur, India (Jahn, 1981) allowed the evaluation of the economic benefits of this alum-saving method. The results are shown in Table 4-1.

The natural polymers studied in India are all prepared in the same manner:

1) The raw material is cleaned of any fibrous material and pulverized.

2) The powder is sieved to remove the husk and is mixed with soda ash at a ratio of 9 to 1.

3) A volume of 0.5 m^3 of water is added to 1.0 kg of the mixture to form a milky solution.

4) The solution is heated (but boiling is not necessary).

The dose is calculated as 1 ml of solution equivalent to 1 mg of coagulant aid. A volume up to 1 m^3 of this solution

(1)

TABLE 4-1: Economic Benefits Achieved by the Use of a Coagulant Aid from Indigenous Plant Material in the Treatment Plant at Kanhan Water Works; Nagpur, India[a]

Total quantity of water treated in July (1970)	1.74 million cubic meters
Total amount of alum conserved	56,040 kg
Value of 56,040 kg of alum at $.09 per kg = 56,040 x .09	$5,000
Labor required for extra operation: 9 laborers per day for 31 days	279 laborers
Cost of labor at $1.20 per laborer	= $330
Power, water, and depreciation of the machinery used for coagulant aid application at $1340 per million cubic meters	= $2,300
Additional cost incurred by using coagulant aid: $330 + $2,300 = $2,630	
Net saving = $5,000 - $2,630	= $2,370
Saving per million cubic meters	$1,360

[a]1982 ENR index = 3729
1970 ENR index = 1385
8.8 rupees = US$1.00

[SOURCE: adapted from Jahn, 1981, p. 182]

can be prepared for use. The solution may be dispensed by conventional solution-type feeders.

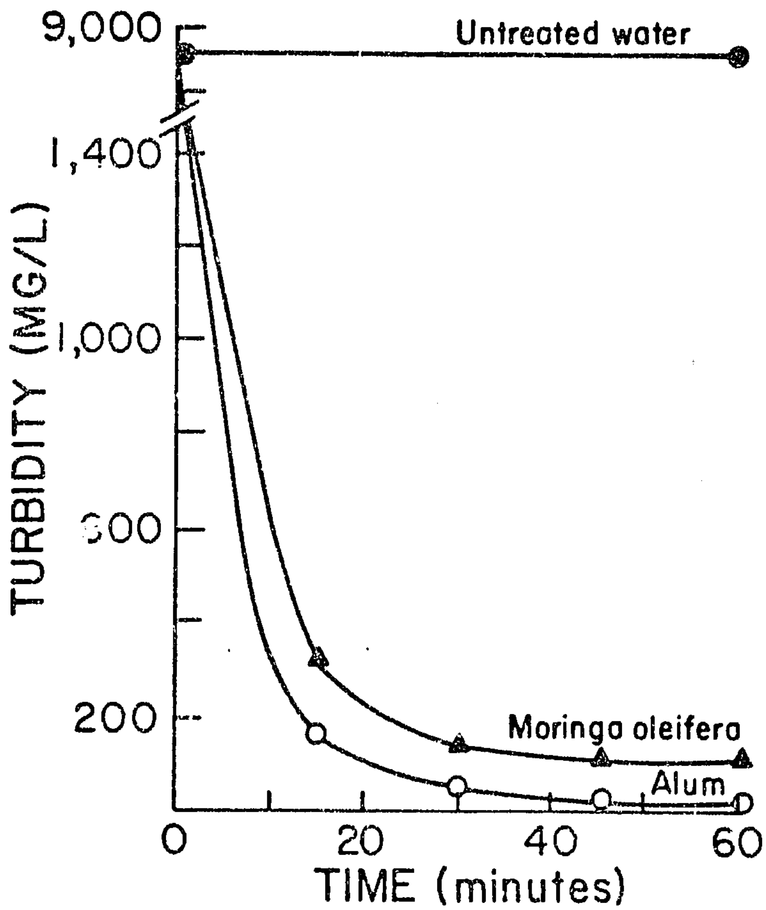
In Peru, two coagulant aids, viz. "Tunafloc A" and "Tunafloc B," have been extracted from indigenous cacti plants (Kirchmer et al., 1975). These two aids, in concentrations of 0.4 mg/l, reduced the raw water turbidity of the Rimac River near Lima from 500 NTU to 70-75 NTU. Furthermore, alum consumption was reduced from 32 to 5 mg/l. These aids are now being applied in community water treatment to reduce alum consumption.

Laboratory studies conducted in the Sudan (Jahn and Dirar, 1979) revealed that seeds from the moringa oleifera tree act as a primary coagulant, and compare favorably with alum with respect to reaction rates and turbidity reductions in the raw water. The results from jar testing showed that alum gave only a further 1% reduction in turbidity, illustrated in the graph of residual turbidity versus settling time in Figure 4-7. The efficiency of several plant materials (including moringa oleifera seeds) as natural coagulants in comparison with alum have been studied experimentally by several investigators. Their work is summarized in Table 4-2.

A remarkable cationic polyelectrolyte, called chitosan, acts faster than any known coagulant from plant materials (Jahn, 1981). It is comprised of deacetylated chiton produced from the exoskeleton of arthropods such as shrimp,

FIGURE 4-7

Coagulating Properties of Moringa oleifera Seeds in Comparison with Alum
(concentration of coagulants: 200 mg/l)



[SOURCE: adapted from Jahn, 1979, p. 92]

TABLE 4-2: Plant Materials Tested in Comparison with Alum as Primary Coagulants and Coagulant Aids for Natural Waters

COUNTRY	SOURCE	TURBIDITY	ALUM DOSE (mg/l)	RESIDUAL TURBIDITY[a] (NTU)	PLANT MAT. AS PRIMARY COAGULANT (mg/ l)	RESIDUAL TURBIDITY[a] (NTU)	PLANT MAT. AS COAGULANT AID (mg/l)	RESIDUAL TURBIDITY[a] (NTU)
				Moringa oleifera (seeds)				
Sudan	Hafir of El Qerabin pH: 8.5	470 NTU	200	11 (1 hr)	200	16 (1 hr)		
Sudan	Nile at Mogren water works pH: 8.4	75 NTU	40	3 (1 hr)	50	10 (1 hr)		
				Moringa stenopetala (seeds)				
Sudan	Nile at Mogren water works pH: 8.4	75 NTU	40	3 (1 hr)	5	8 (1 hr)		
				Maerua pseudopetalosa (root)				
Sudan	Hafir of El Qerabin pH: 8.5	470 NTU	200	11 (1 hr)	200 (1 hr)	35 (1 hr)		
				Strychnos potatorum (seeds)				
India	Yamuna River pH: 8.2	2200 NTU	65	40 (2 hrs)	3-3.5	16-17	1, followed after 3 min by 10-15 mg/l of alum	8.5-1.5 (0.5 hr)
	Canal Water	500 NTU			2	30 (1 hr)		
				Opuntia ficus indica (sap extracts)				
Peru	Rio Rimac	28 NTU	32	5 (0.5 hr)			0.4 + 0.5 mg/l of alum	14-15 (0.5 hr)

[a]The settling time following coagulation is indicated directly below the turbidity results.

[SOURCE: adapted from Jahn, 1981, p. 170]

prawns, and lobster. Approximately 400 lbs. of chitosan can be derived from a ton of shrimp meal. Table 4-3 summarizes the results of laboratory tests on chitosan conducted in India. Jar tests were run for both alum and chitosan using: (1) flash mix for one minute at 100 rpm; (2) flocculation for 9 minutes at 40 rpm; and (3) settling for 10 minutes. The results show that the coagulation properties of chitosan surpass those of alum at high turbidities. Moreover, as a coagulant aid, chitosan can effect an even greater reduction in turbidity than using either primary coagulant alone. Further research is being conducted in India by NEERI on the effectiveness of fish scales and bones as coagulant aids.

A potentially deleterious side-effect of some natural polyelectrolytes is their propensity for increasing the growth of bacteria in the water being treated. Independent studies conducted in India and the Sudan showed that seeds from the nirmali and moringa trees, when used as coagulant aids, initially removed bacteria from the water, but after several hours the bacteria count rose slightly (Jahn, 1979). This phenomenon was attributed to the organic material present in the seeds which was thought to provide additional substrate for the growth of bacteria. At any rate, proper disinfection of the treated water will kill microbiological organisms, including bacteria.

Other potential problems associated with natural polyelectrolytes are their widespread use as foodstuffs

TABLE 4-3: Efficiency of Chitosan as a Primary Coagulant and a Coagulant Aid

RAW WATER TURBIDITY (NTU)	ALUM (MG/L)	RESIDUAL TURBIDITY (NTU)	CHITOSAN AS PRIMARY COAGULANT (MG/L)	RESIDUAL TURBIDITY (NTU)	CHITOSAN AS COAGULANT AID (MG/L)	RESIDUAL TURBIDITY (NTU)
3200	300	90	1.00	10	0.15 + 20 mg/l alum	4
1400	100	10	1.00	10	0.1 + 20 mg/l alum	3
500	30	5	0.25	25	0.1 + 5 mg/l alum	5
70	10	14	0.25	18	0.05 + 8 mg/l alum	10

[SOURCE: Jahn, 1981, p. 185]

which may make them difficult to procure without causing local scarcity; and their quality tends to deteriorate in time, and, therefore they should not be stored over 3 months.

Disinfection

The disinfection of potable water supplies is almost universally accomplished by the use of chlorine gas or chlorine compounds (hypochlorites). Their ability to kill pathogenic organisms and to maintain a residual in the distribution system, as well as their wide availability and moderate cost in most regions of the world, make them well suited for disinfection. Presently, the only viable alternative to chlorination for the disinfection of community water supplies is ozonation, which has been increasingly used in European water supplies. However, the use of ozone is not generally recommended for developing countries, due to the high installation and operating and maintenance costs of ozonation, the need for a continuous supply of power, and the need for importation of the equipment and spare parts.

The period available for the interaction between the disinfectant and constituents in the water, called the contact time, is important in the design of disinfection systems for water treatment. The minimum contact time for chlorination should be 10 to 15 minutes to ensure effective disinfection (Cox, 1964), which can generally be provided in the transmission main before the first consumer. Should such contact not be available, a contact chamber may be used. In filter plants, the clear well may be designed to provide the contact through baffles to avoid short-circuiting.

The decision to use either chlorine gas or hypochlorites should be based on several factors: (1) the quantity of water to be treated; (2) the cost and availability of chemicals; (3) the equipment needed for its application; and (4) the skill required for operation and control. Chlorine gas feed equipment is more expensive, more difficult to operate and maintain, and more dangerous than solution-type hypochlorinators; and in most instances has to be imported. On the other hand, chlorine gas is generally much less expensive than hypochlorite containing an equivalent amount of available chlorine, and can be stored for longer periods of time without deteriorating. The convenience and economy of using chlorine gas, then, is counterbalanced by cost and complexity of gas chlorinators as well as safety requirements. Furthermore, chlorine gas is often only available in the capital cities of developing countries; hence, its cost in outlying communities may increase proportionally to the shipping distance from the capital city. A study conducted in Brazil (Macedo and Noguti, 1978) has shown chlorine gas to be more economical than sodium hypochlorite for plants with capacities greater than 500 cubic meters per day (see Chapter 10). The study compared chemical, transportation, installation, operation, and maintenance costs for the two chemicals. The final choice to use either chlorine gas or hypochlorite compounds can only be made after considering each installation on an individual basis.

A Handbook of Chlorination by G. C. White (1972) covers the practical and theoretical aspects of both chlorination and hypochlorination. It is widely recognized as the best single reference on chlorination.

Gaseous Chlorine

Chlorine is a greenish-yellowish gas which under pressure is converted to liquid form. Liquid chlorine may be purchased in pressurized steel cylinders which are furnished in various sizes from 40 to 100 kg to 900 kg (about 1 ton). The cylinders are so filled that liquid chlorine fills 80% of the capacity at a temperature of 65°C. Such a procedure safeguards against possible ruptures of the cylinders at high temperatures and pressure. The flow of chlorine gas from a container depends on the internal pressure, which in turn depends on the temperature of the liquid chlorine. The normal discharge rate for a 50 kg cylinder at 21°C is about 800 gm/hour against a 240 KPa back pressure; the discharge rate for a 900 kg cylinder is about 7 kg/hour under similar conditions. If chlorine demand requires the use of several cylinders, manifolding the cylinders to one chlorinator is preferred over providing separate feeders for each cylinder.

Chlorine cylinders should be placed on weighing scales, each scale holding one cylinder, to provide uninterrupted service while empty cylinders are being replaced. The weights of the cylinders should be recorded at regular intervals, at least once a day, in order to ascertain the actual quantity of chlorine being used, which serves as a check on the accuracy of the dosing.

Chlorine gas is dangerous and corrosive and care must be exercised in handling containers. Heat should never be applied to chlorine containers or valves. Containers should be stored in a dry location and protected from external heat. In tropical countries, it is important that chlorine cylinders and feeding equipment be shielded from the direct rays of the sun. The storage area should be outdoors or, if indoors, well ventilated, preferably by forced ventilation. The containers should be stored and used in the order in which they are received. Gas masks designed to protect against chlorine fumes should be available. Safety requirements for the handling and feeding of chlorine gas are given in White's Handbook (1972).

Hypolchlorite Compounds

Calcium, sodium hypochlorites, and chlorinated lime (bleaching powder) are the compounds commonly used. Their chemical and physical characteristics are listed in the Table of Chemicals, found in Appendix A. The available chlorine in these compounds varies from 10 to 15% for sodium hypochlorite, to 33 to 37% for bleaching powders, to 70% for "high-test" calcium hypochlorites. Bleaching powders are relatively unstable; exposure to air, light, and moisture makes the chlorine content fall rapidly. High-test hypochlorites are considerably more stable; under normal conditions they will lose 3 to 5% available chlorine per year. The properties, storage, and preparation of hypochlorite solutions are covered in several standard texts (White, 1972; Cox, 1964; AWWA, 1971).

The choice of hypochlorite for a particular installation should be based on cost and availability. Other factors being equal, the chemical of choice would be sodium hypochlorite, which is easily fed and poses no fire hazard when stored. In remote areas, though, where chemicals must be imported, calcium hypochlorites are more economical because the Cl_2 content is greater.

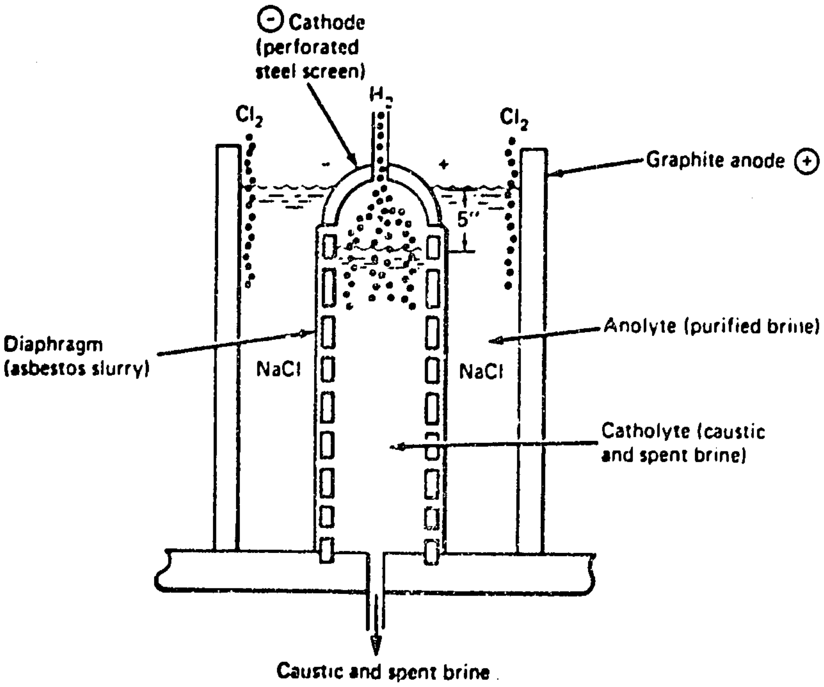
<u>On-Site Manufacture of Disinfectant</u>

In small communities and villages in remote areas of the world, where the proper handling, transporation, and storage of lethal chlorine gas and highly reactive chemical compounds cannot be assured, the local production of chlorine may be more practical than its importation. Chlorine is commercially manufactured by the electrolysis of brine, primarily in diaphragm cells. Chlorine comes off as a hot gas at the anode, and hydrogen comes off at the cathode together with a co-product, caustic, resulting from the electrolysis of the brine solution (White, 1972). A diaphragm separates the catalyte from the anolyte. Figure 4-8 shows a schematic of a typical diaphragm cell. The raw materials required for on-site hypochlorite generation are brine, water, and electric power. The brine and water should be relatively free from impurities (e.g. Ca, Mg, Fe) to minimize blockage in the cell diaphragm. Also, high ammonia-nitrogen concentrations in the brine water should be avoided, as this can lead to the accumulation of nitrogen trichloride (NCl_3) in the chlorine cylinders, an extremely volatile substance that can suddenly explode and rupture the cylinders.

The power and salt requirements for several American proprietary devices for on-site hypochlorite generation are listed in Table 4-4. At power costs of 5 cents per Kw-hr and salt costs of $10 per ton, the costs for power and salt are 916 to 14 cents per pound of sodium hypochlorite. To this, of course, must be added electrode replacement costs and operating labor costs. Total costs are substantially less than purchased hypochlorite and compare favorably with liquid chlorine for smaller installations. For example, the total cost of the "Ionics" hypochlorite generation systems for 1000 lb/day production is estimated at 10 cents (7.4

FIGURE 4-8

Schematic Diaphragm Cell for Chlorine Generation



[SOURCE: White, 1972, p. 6]

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TABLE 4-4: Power and Salt Requirements for On-Site Hypochlorite Generation as Reported by US Manufacturers

	Power kw hr/lb Na	Salt lbs/lb NaOCl
Ionics "Cloromat"	1.7	2.1
Engelhard "Chloropac"	2.8	(Sea water)
Pepcon "Pep-Clor"	3.5	3.25
Diamond Shamrock "Sanilec"	2.5	3.05

[SOURCE: Culp and Culp, 1974, p. 195]

operating and 2.6 capital) per lb of available chlorine; whereas the total cost of truck-delivered hypochlorite is about 30 to 45 cents per lb of available chlorine and liquid chlorine costs 9 to 15 cents per lb in ton cylinders (Culp and Culp, 1974).

The electrolyte process described briefly above has been adapted for producing sodium hypochlorites on-site in small communities in Russia (IRC, 1977a). The installations are capable of producing 1 to 100 kg of active chlorine per 24 hours, and are composed of graphite electrodes that are claimed to be simple, reliable, and safe.

A design that is similar to the Russian units has been adopted by the Intermediate Technology Development Group in England for the construction of a pilot electrolysis unit for a treatment plant in the town of Beira, Mozambique (Intermediate Technology Services, 1982). Each module is rated at 5 Kg/day of chlorine equivalent. The cell is operated by a 0-8 volt direct current supply from a transformer rectifier unit connected to a standard 220/250 volt power supply. All materials are PVC plastic or titanium, and are corrosion-resistant. The manufacturer of the unit, Ecological Engineering Ltd. of the United Kingdom, claims that it can function (1) on untreated water and independently of water pressure; (2) with impure salt; (3) on fluctuating electrical voltage, and (4) with low maintenance costs. The pilot unit was installed in 1981 at a cost of L5000 (US$8600); therefore it would have to operate for a period of two years to pay for its foreign exchange costs in terms of calcium hypochlorite or 3½ years in terms of gaseous chlorine (based on 1982 prices in Mozambique). It should be noted that this was a single-purchase, and that unit costs could be reduced substantially if a country-wide program of on-site hypochlorite generation was initiated. The pilot project has encountered several problems so far which are summarized below. More information on this particular project may be obtained from the Companhia das Aguas de Beira (Water Authority for Beira, Mozambique).

1) The production of active chlorine begins to decrease the rated capacity of the generator when the ambient temperature is greater than 30^{o}C (cooling of the unit will increase its efficiency, but this would substantially complicate the unit);

2) There is a need for a simple method to check the chlorine concentration of the solution (colorimetry methods have been used, but these require careful dilution); and

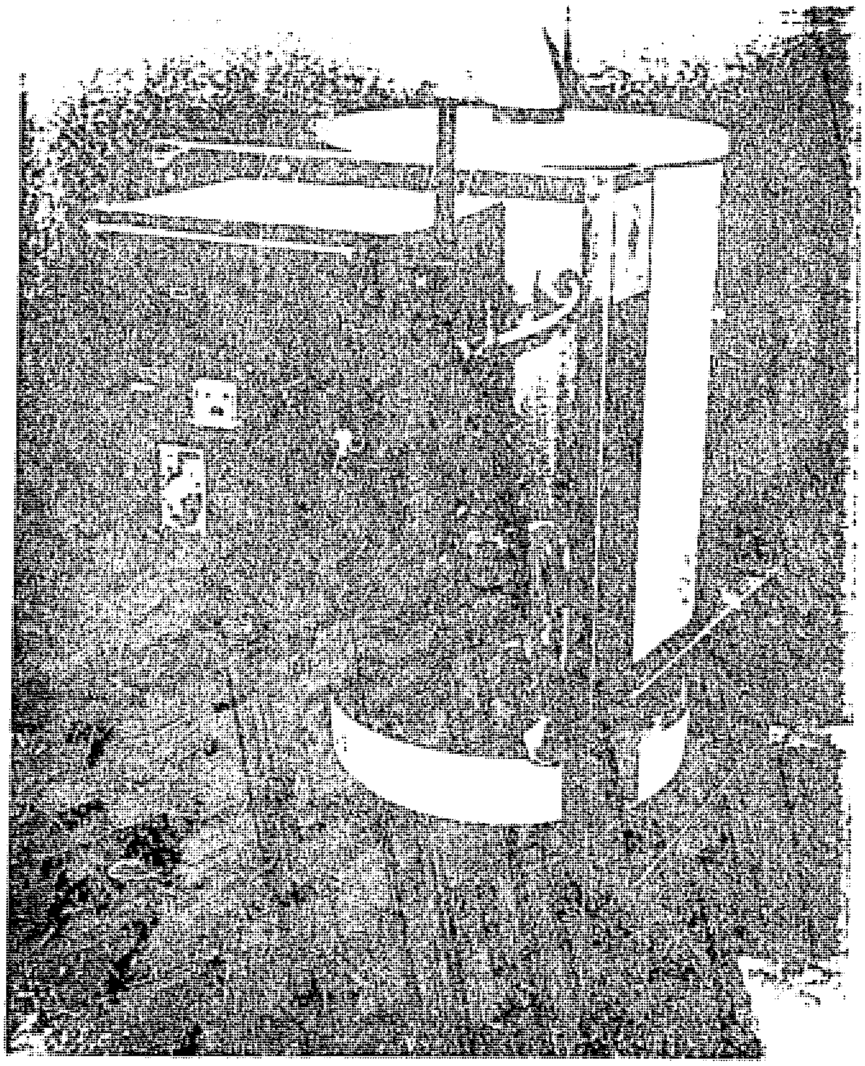
3) For larger plants, the volume of tankage required becomes excessive (2000 liters are required to produce 5 kg/day), although several units could be operated in parallel, if this is considered feasible economically.

An alternate approach to conventional electrolytic cells, which are susceptible to clogging when impure brine solutions are used, are non-diaphragm cell systems that produce nascent sodium hypochlorite from a weak brine solution. Unlike diaphragm or membrane cells which separate the products of electrolysis (Cl_2, NaOH, and H_2), no chlorine gas is produced with non-diaphragm cell systems. Instead, the various components are allowed to react within the cell to produce a relatively dilute solution of NaOCl in the 0.5 to 0.8% range (5 to 8 gr/liter). Also, research conducted by SIENCO, Inc., St. Louis, Missouri (1982) has shown that for any given dosage level, nascent hypochlorite is a more effective disinfectant than commercial sodium hypochlorite (or bleaching powder). This was attributed to the absence of excessive caustic which is contained in all commercially produced solutions, and is generally believed to rob commercial hypochlorite of most of its bacteriocidal properties.

Figure 4-9 shows a nascent sodium hypochlorite generator manufactured by SIENCO, Inc. This unit is intended for water treatment in villages and small towns with populations up to approximately 5,000 people. One of the unique features of this system is that it is totally devoid of pumps and other feed mechanisms which are standard equipment in most conventional chlorinating devices. The system utilizes the gas lift and heating effect of the electrolysis to circulate the solution in a continuous closed-loop mode. The unit is capable of producing 350

FIGURE 4-9

Nascent Sodium Hypochlorite Generator
(manufactured by SIENCO, Inc.)



[SOURCE: SIENCO, Inc., personal communication]

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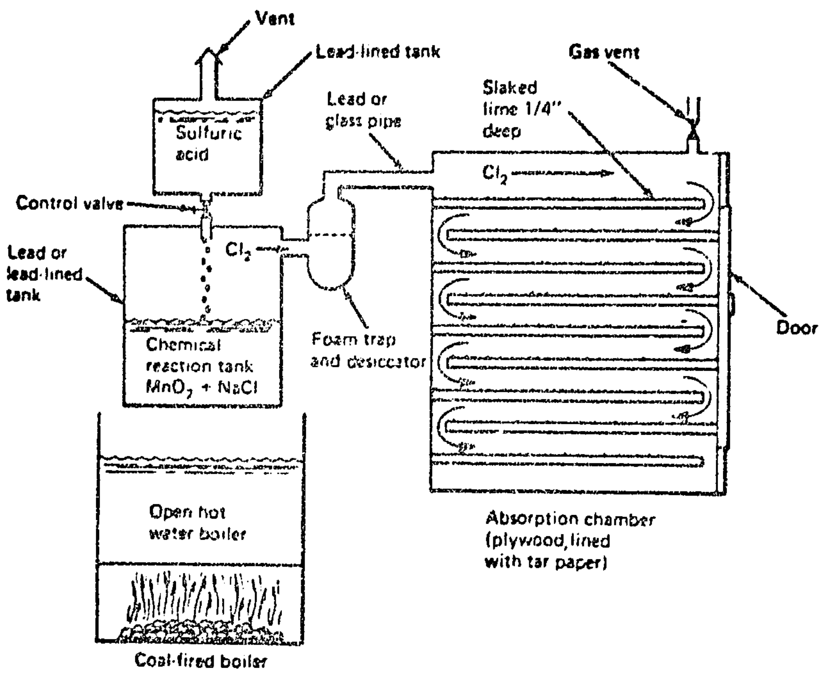
liters of sodium hypochlorite solution in a 12 hour period, at a concentration between 1500 and 2000 mg/l (expressed as equivalent chlorine). The only moving part in the system is the mechanical timer located on the front of the Control Cabinet. If impurities are present in the brine solution, a slime layer may build up on the inside of the electrolytic cell, but this is easily removed by applying an acidic solution that is provided by the manufacturer. The cost of the unit is about $3000. This unit has been used in Africa and Asia and is currently being introduced into several Latin American countries.

An alternative to the electrolytic production of chlorine was developed by Stone in 1950, while working in the interior of central China, and functions without electric power. The raw materials employed were common salt, manganese dioxide, sulfuric acid, and low-grade slaked lime, all of which were manufactured or mined within the region. By the direct chemical combination of chlorine gas that is produced by the process with the slaked lime, it was possible to make bleaching powder with about 35% available chlorine. A schematic of the installation is diagrammed in Figure 4-10. The two principal sections are the chlorine gas generator and the absorption chamber. In the generator, the furnace heats the water which, in turn, gently warms the chemicals placed in the tank, accelerating the reaction. The amount of chlorine gas that is generated is controlled

FIGURE 4-10

Generation of Hypochlorite without using Electricity



[SOURCE: White, 1972, p. 646]

by the heating operation as well as by the addition of sulfuric acid, whose feed-rate is regulated by a control valve. The chlorine gas is passed through a foam trap and a dessicator, where it is dried before going to the absorption chamber. The absorption chamber consists of several trays on which a 0.6 cm layer of slaked lime is placed. The chlorine gas enters at the top of the chamber, and being heavier than air, circulates downward and reacts with the slaked lime to form bleaching powder. It was possible to produce about 14 kg of bleaching powder (at 35 to 39% chlorine strength) after 12 hours of operation. The physical dimensions of this design could be easily enlarged to increase its capacity for larger sized plants.

Chemical Feeding

Chemical feeders should be simple in design and easy to operate. Hypochlorite and coagulant solutions may be fed by simple solution-type feeders that are constructed locally. Dry-chemical feeders are somewhat more complex and have greater maintenance problems. As chlorine gas feeders are more complex than solution-type feeders, so their use is limited to larger plants where skilled supervision is available and the economy of using chlorine gas counterbalances the disadvantages. Lime may be fed either as a continuously-mixed suspension by slurry-type feeders, or as a solution by saturation towers.

Chlorine Feeding

Chlorine gas is applied by two distinct methods: solution-feed and direct-feed. In both instances, the chlorine gas is obtained from the evaporation of liquid chlorine maintained under pressure in steel cylinders. Dry chlorine liquid or gas is non-corrosive and can be stored safely in steel cylinders and transmitted safely by black or wrought-iron piping. By contrast, chlorine solution and liquid or gaseous chlorine containing moisture or in humid atmospheres are highly corrosive and must be transmitted by piping constructed of silver, glass, hard rubber, plastic, or other materials of proven resistance (Fair, Geyer, and Okun, 1968).

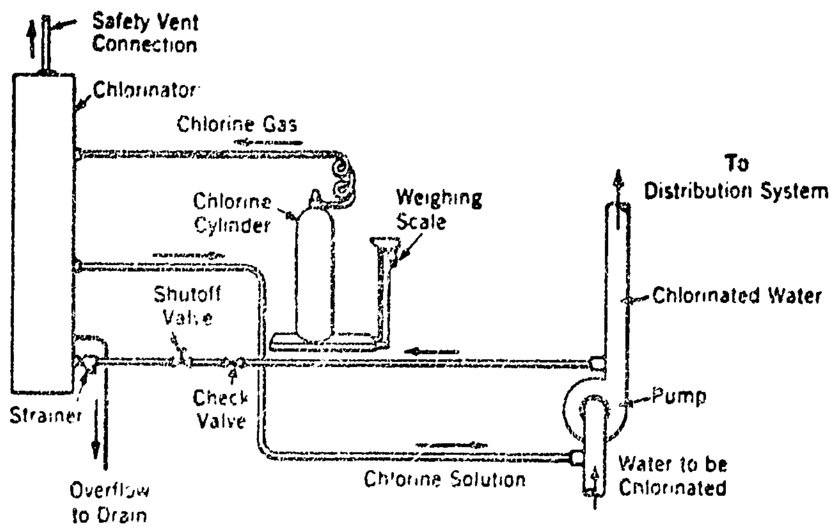
Solution-Feed Chlorinators

Solution-feed chlorinators take gaseous chlorine evaporated in the container, meter it, and mix it with water to form a strong chlorine solution. Distribution-system-pressure water is often used to inject the chlorine solution into the water supply. Water under pressure while passing through a venturi tube or orifice creates a negative pressure which draws the chlorine gas into the chlorinator. The water pressure should be 170 KPa or at least three times greater than that of the water being chlorinated (whichever is higher) to ensure effective injection. This is preferred over using separate pumping installations that require electric power. A schematic for a chlorinator tied to the high-service pumping is shown in Figure 4-11. In general, the pressure difference between the discharge side of the high-service pumps, which supplies water to the chlorinator, and the suction side of the pumps, where the chlorine-water solution is injected, is more than adequate for proper injection. Also, by using the high-service pumping for chlorine injection, it is unlikely that finished water would leave the plant unchlorinated; inasmuch as the stoppage of those pumps due to mechanical failure or power outages would halt both chlorination and the flow of water out of the plant.

An "all-vacuum" system for feeding chlorine is shown in Figure 4-12 (Capital Controls Co. brochure). The operation of the system is relatively simple. Water under pressure (e.g. from the discharge side of the high-service pump) passes through the injector at high velocity causing a negative pressure. The negative pressure is transmitted through a plastic tube to the remote chlorine flow meter, and then through another tube to the chlorinator which is mounted on the chlorine cylinder. With negative pressure at the chlorinator, a spring-opposed diaphragm opens the chlorine safety valve at the inlet of the chlorinator. Chlorine at cylinder pressure enters through the inlet valve where the pressure is reduced below atmospheric pressure. The gaseous chlorine is then conducted to the meter unit where the flowrate is measured, and where a spring-opposed

FIGURE 4-11

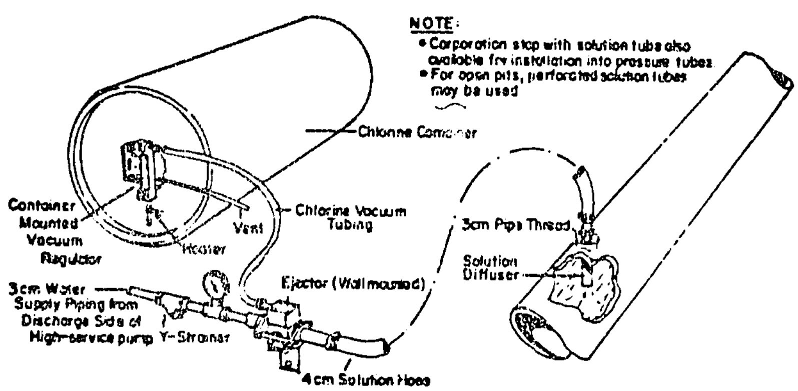
Chlorinator Installation using Pressure from High-service Pumping



[SOURCE: Hardenbergh and Rodie, 1961, p. 280]

FIGURE 4-12

"All-Vacuum" System for Feeding Chlorine



[SOURCE: Capital Controls Co. brochure]

diaphragm regulates the negative pressure in the system, which is held constant regardless of cylinder pressure. Finally, the gaseous chlorine passes under negative pressure to the injector where it is injected as a chlorine-water solution.

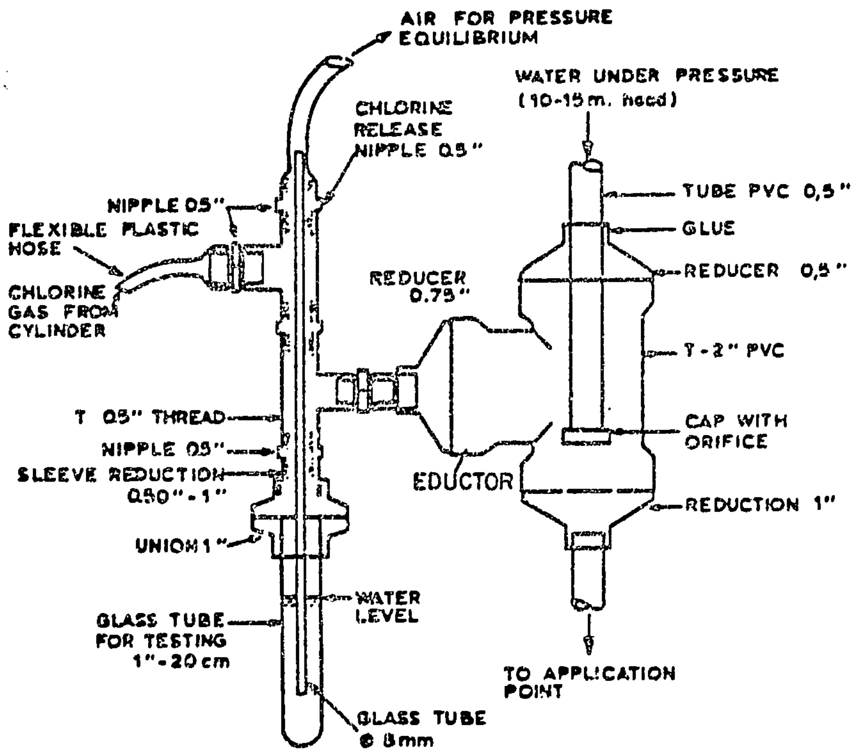
A locally constructed chlorinator, built from plastic and glass for about US$15, has been used in Brazil (IRC, 1977a), and is shown in Figure 4-13. The chlorine gas is introduced into the water by the negative pressure created in the injector. The depth of water in the glass test tube is a measure of that negative pressure. The dose can be regulated by adjusten the auxiliary valve on the cvlinder or reducing the water flow in the injector. Unlike the "all-vacuum" system described previously, the negative pressure in this system cannot be constantly maintained.

Direct-Gas Feed Chlorinators

Direct-gas feed chlorinators feed the chlorine gas through a diffuser directly into the water, to be treated by utilizing the pressure of the chlorine in the container. This unit is suited for use where chlorination is required, in the absence of either electricity to run a booster pump or a pressured water supply offering a sufficient differential for a solution-feed chlorinator. A diffuser made of silver or porous stone is needed because of the corrosive nature of chlorine gas in water. Direct-feed units are more effective in warm climates (above 10°C); at

FIGURE 4-13

Low-cost Chlorinator Fabricated in Brazil



[SOURCE: IRC, 1977a, p. 360]

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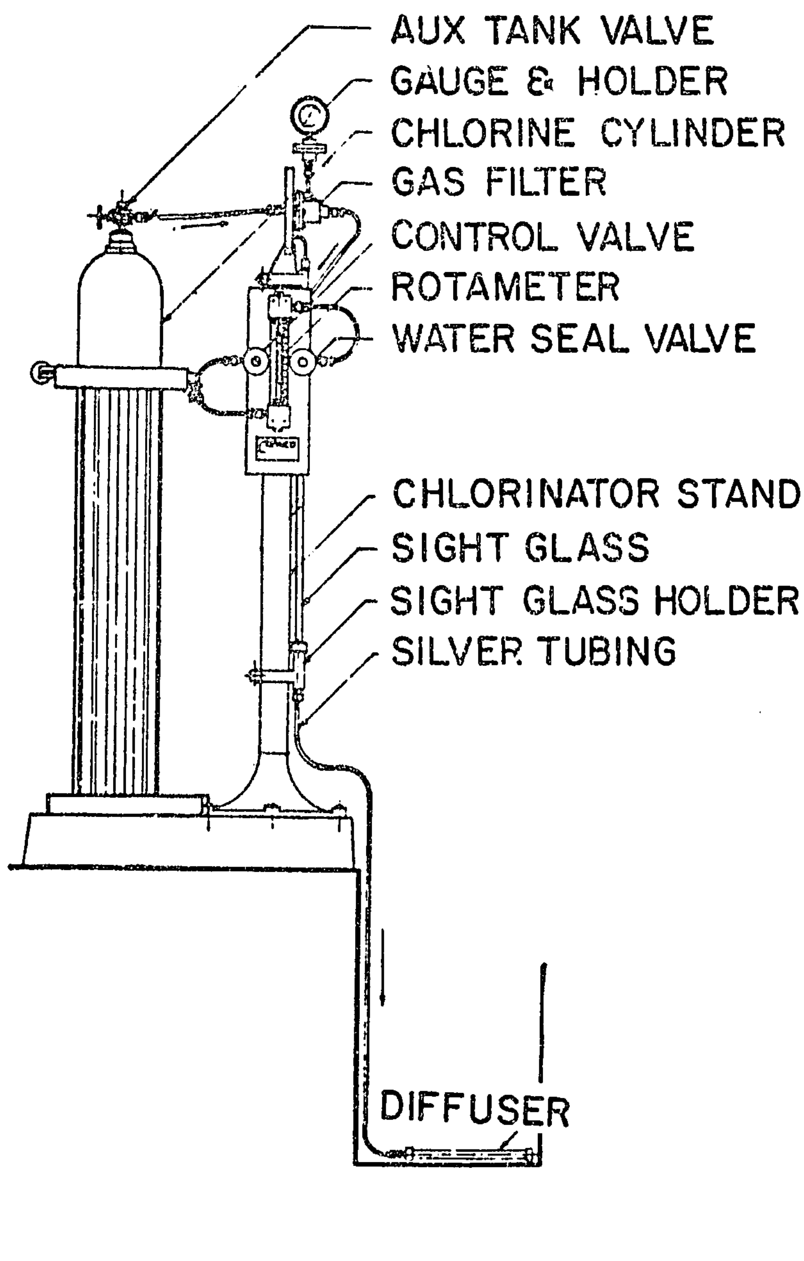
lower temperatures, chlorine combines with water to form chlorine hydrate (commonly called chlorine ice). The hydrate may obstruct feeding equipment. The maximum capacity of individual direct-feed units is about 35 kg per day, and 135 kg per day when applied to a pipeline or main, and to an open tank or channel, respectively (ASCE, 1969). A simple direct-feed gas chlorinator is shown in Figure 4-14. Although chlorine gas dosage is not regulated as accurately as with solution-feeders (about 4% error in flowrate can be expected), direct-feed chlorinators are relatively simple devices that may be appropriate for smaller communities.

Chlorine solutions are discharged into either pressure conduits or open channels. Typical chlorine diffusers that are used for each situation are illustrated in Figures 4-15 and 4-16. They are fabricated from plastic pipe or similar corrosion-resistant material. The materials used in pipelines should always be designed to introduce the chlorine solution into the center of the pipe. For channels, the chlorine solution should not be applied at a depth less than 50 cm as it will not be completely absorbed. Hydraulic jumps or baffled channels are suitable mixing devices for open channels. The chlorine diffuser should be placed immediately upstream of the point of turbulence, as shown in Figure 4-16.

FIGURE 4-14

Direct-feed Gas Chlorinator



[SOURCE: UNC International Programs Library, personal communication]

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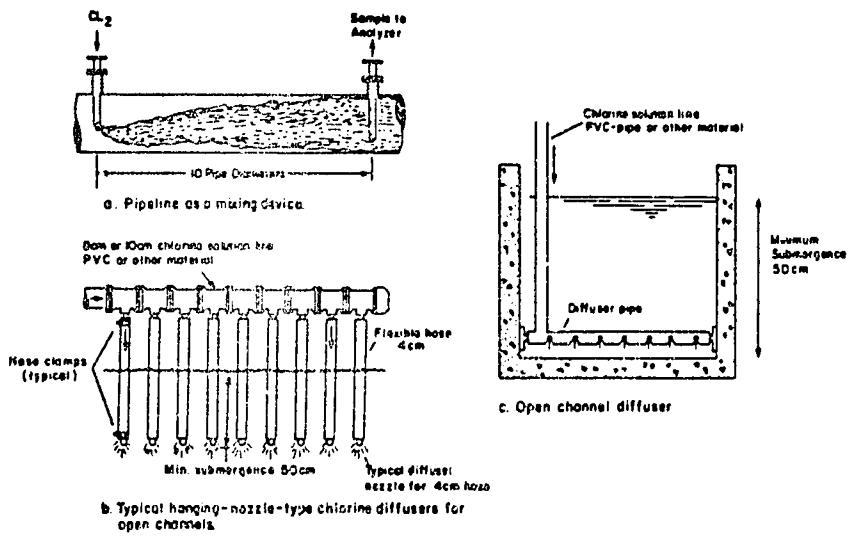
FIGURE 4-15

Typical Designs for Chlorine Diffusers
(installed at the point of chlorine application)



[SOURCE: White, 1972, pp. 136-137]

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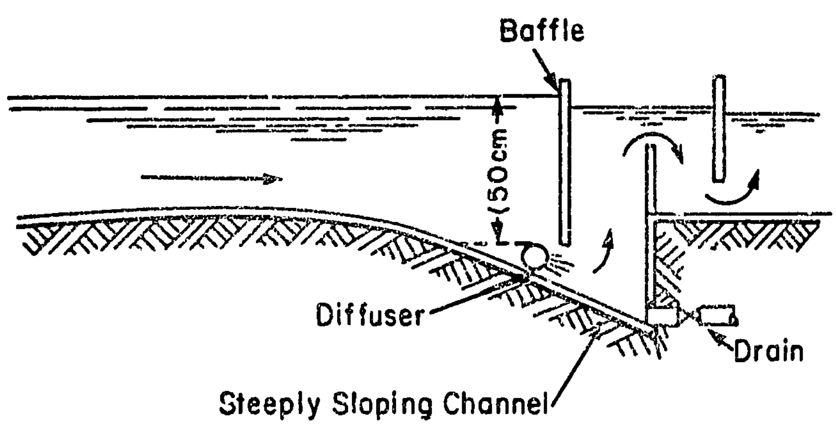
FIGURE 4-16

Baffled Mixing Chamber for Chlorine Application in Open Channels



[SOURCE: Azevedo-Netto, personal communication]

Solution-Type Feeders

Solution-type feeders apply to both hypochlorite and coagulant solutions. Some general requirements for the design of these feeders are:

1) a minimum of 2 chemical feeders provided at each application point, or a total of 3 if two chemicals are to be fed, so that feeding is not interrupted when one unit is out of service;

2) the combined capacity of the chemical feeders should be greater than the maximum dosage required, but not so large as to be inaccurate during periods of low flow;

3) concrete tanks adequately protected against corrosion by a layer of bitumastic enamel, plastic, rubber, or a similar-functioning substance;

4) adequately-sized drains to facilitate the cleaning of the tanks and the flushing out of the accumulated sediments;

5) provisions for either hand-operated or motor driven paddles for mixing the chemical solutions (except for saturation towers);

6) chemical feed lines made from rubber or plastic hose supported at short intervals, or placed inside a tile pipe for protection, with provision for easy cleaning. Open channels are best for conveying lime suspensions, albeit often impractical.

7) the location of chemical feeders as close as possible to the point of the application of the chemicals.

Solution-type chemical feeders may be classified into three groups: (1) constant-rate; (2) proportional; and (3) saturation towers. Some designs that are practical for developing countries are described briefly here, more detailed information and additional designs are given elsewhere (AID UNC/IPSED, 1966; Arboleda, 1973).

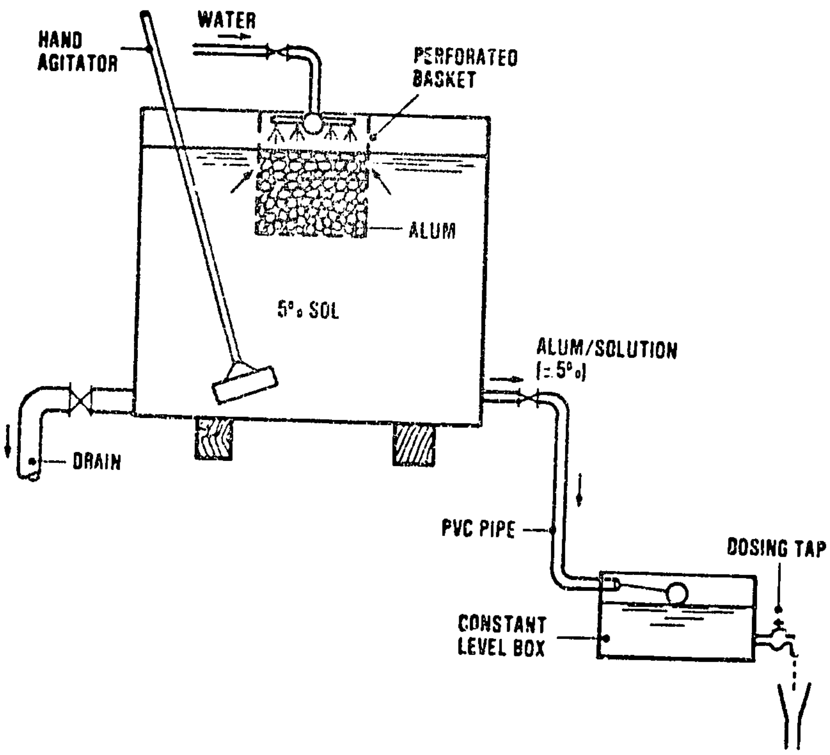
Constant-Rate Feeders

The constant-head solution feeder has been used widely for chemical feeding in water treatment plants. A typical arrangement for feeding alum solutions is shown in Figure 4-17. The feeding system consists essentially of (1) a solution tank for dissolving and/or mixing the chemicals in water to produce solutions of known strength; (2) a constant-head box with a float-valve inlet; and (3) a dosing tap or adjustable orifice attached to the outlet side of the constant-head box for adjusting the flow of chemical solution. All exposed parts should be protected against corrosion, including the float which can be made of hard rubber, glass, or ceramic. Two solution tanks can be connected to a single constant-head box to assure uninterrupted operation while one tank is being filled.

A simple chemical feeder that functions without a constant-head box is a constant-head orifice feeder in which the orifice may be adjustable. Two different designs for a hypochlorinator and alum doser are shown in Figure 4-18 and 4-19. The rate of feed is controlled by maintaining a

FIGURE 4-17

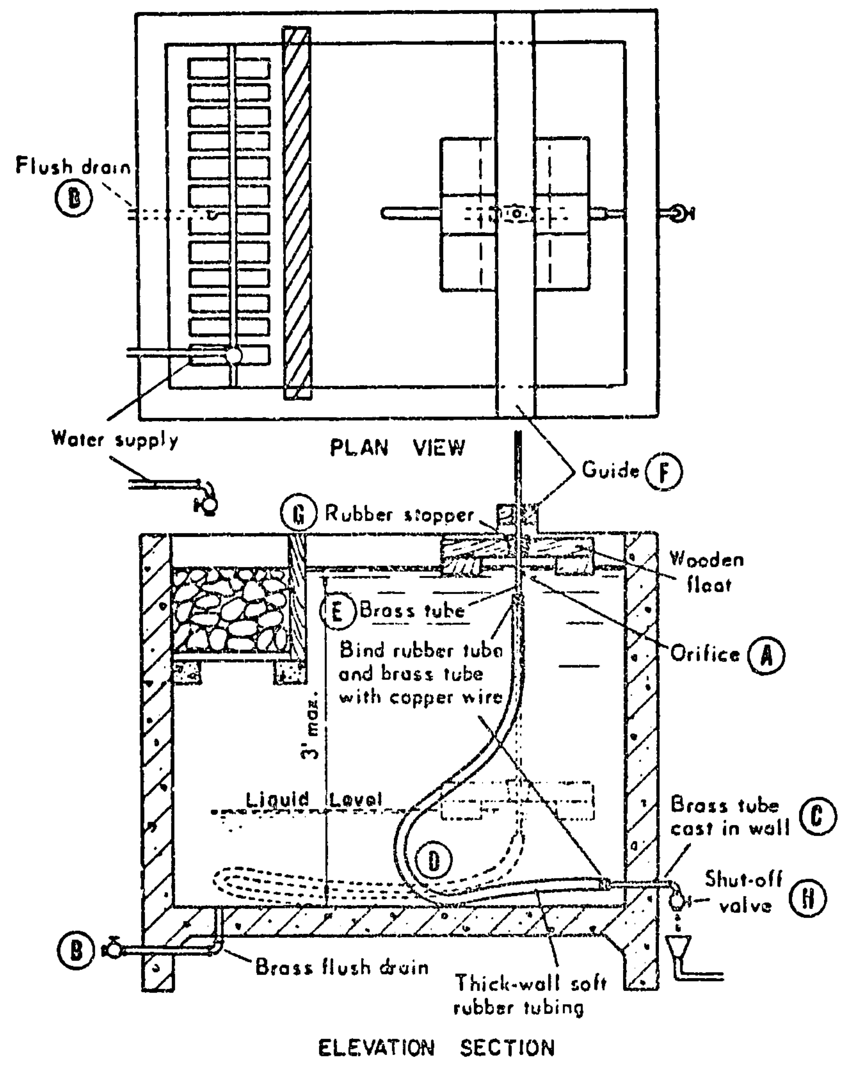
Constant-Head Solution Feeder for Alum Dosing



[SOURCE: IRC, 1981b, p. 200]

FIGURE 4-18

Floating-arm Type Alum Solution Feeder



[SOURCE: AID UNC/IPSED, 1966a]

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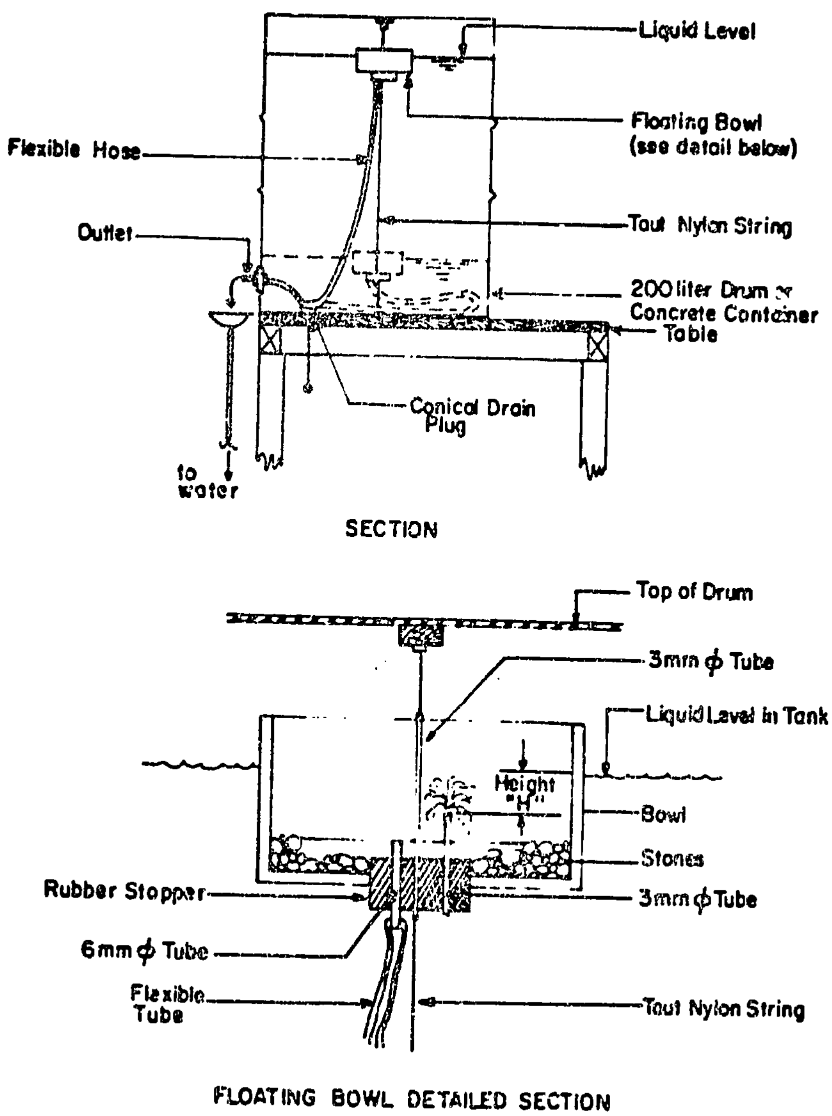
FIGURE 4-19

Floating-Bowl Type Hypochlorinator



[SOURCE: adapted from AID UNC/IPSED, 1966b] constant pressure head on the discharge orifice by floating it at a constant depth below the liquid surface. The designs differ in the method by which the constant head is maintained and the type of discharge orifice employed. These chemical feeders are easily converted to alum dosers or hypochlorinators by simply adding or removing the alum tray and perforated-water pipe in the tank (which is shown in the alum doser in Figure 4-18).

These designs have distinct advantages over conventional constant-head tanks in that they do not require float valves which are difficult to keep clean and subject to corrosion. Moreover, they have been proven from experience to be more accurate in controlling the rate of flow than either control valves or petcocks. The feed rates should be calibrated for each unit after installation, and a chart provided for ready reference.

Another type of constant-rate feeder is the rotating dipper, which consists of a tank in which calibrated cups revolve into the solution and remove adjustable volumes of solution. Such a unit may be powered by a small electric motor or by water flow, as described below. Rotating dippers are especially suitable for feeding lime suspensions, which must be agitated continuously, and hence, cannot be fed by constant-head type solution feeders.

PROPORTIONAL FEEDERS

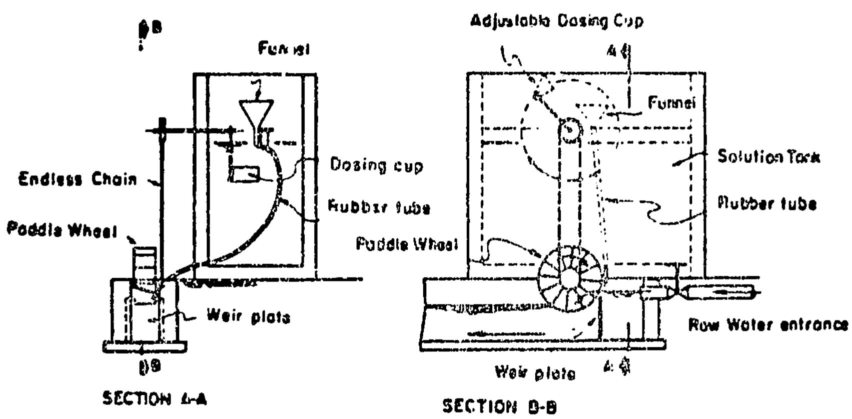
A water powered version of the rotating dipper that feeds chemicals proportional to the raw water inflow was developed in Swaziland (AID-UNC/IPSED Series Item No. 5, 1966) and is illustrated in Figure 4-20. The flow in the influent channel drives a paddle wheel at a speed proportional to the flow rate. The wheel drives a shaft, which in turn rotates the dosing cups that are attached. The dosing cups are located in two solution tanks located on either side of the raw water channel. The cups sweep through the chemical solution in the tank and each cup empties a fixed volume of solution into a funnel, which conveys the solution into the channel immediately upstream of the paddle through a rubber tube.

A chemical dosing unit that operates hydraulically from distribution-system pressure was developed recently by Wallace and Tiernan, Ltd. (Richmond, 1981). The assembly is shown for two operating cycles in Figure 4-21. It consists of three basic components: (1) a water tank of approximately 150 liter capacity; (2) a hydraulically powered adjustable dosing head with an integral 10 liter capacity chemical reservoir, and (3) a quick acting, high capacity syphon.

The principle of operation is simple: water enters the 150-liter tank at any rate between 2.3 m^3/day and 100 m^3/day. As the level in the water tank rises, a ball float

FIGURE 4-20

Proportional Chemical Feeder
(developed in Swaziland)



[SOURCE: Arboleda, 1973, p. 97]

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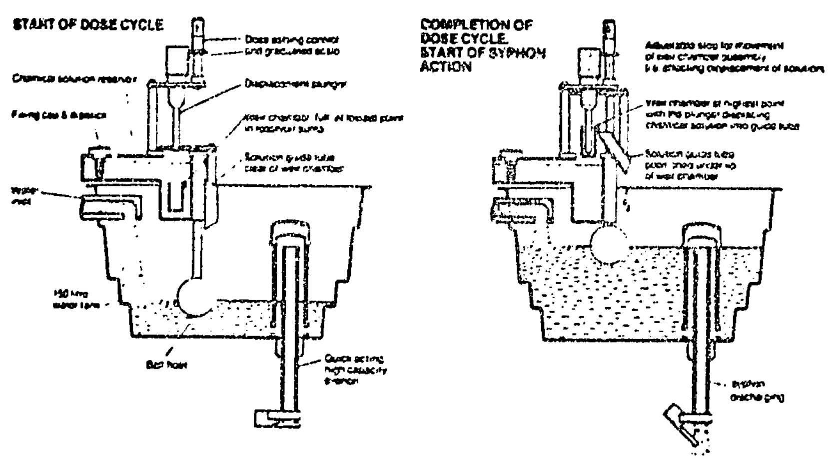
FIGURE 4-21

Hydraulically Operated Chemical Solution Feeder
. (manufactured by Wallace and Tiernon, Ltd)



[SOURCE: Richmond, 1981]

(37)

lifts a small container or weir chamber, full of sodium hypochlorite, to a point where a fixed plunger enters the chamber to a pre-set variable depth and in so doing, displaces a controlled amount of the contents. The displaced hypochlorite is directed into the main tank by means of a special pivoted tube, which moves clear when the chamber descends into the hypochlorite reservoir where it is replenished in readiness to the next cycle. As the water in the main tank reaches an appropriate level, the syphon actuates and discharges the contents of the tank into a service reservoir or tank immediately below the unit.

At maximum dosage and water flowrate, the 10 liter capacity chemical reservoir will last for about 3 days without replenishment. Because the unit is based on cyclic operation within the range of the machine, the chemical dose is proportioned to the incoming flow of water.

Saturation Towers

The use of saturation towers makes it possible to use inexpensive chemical compounds of low purity which may be available locally. However, when used in small treatment plants treating flows less than 1000 m^3/day, the dosage may not be precise. Saturation towers have been used effectively to feed alum-cake (low-grade unpurified alum in large lumps or blocks) in large water treatment plants (AID-UNC/IPSED Series Item No. 12, 1967; Arboleda, 1973). The alum-cake is stored in a tower similar to that shown in

Figure 4-22. The tower can be made of wood or other material. An overhead spray spreads water over the top of the tower. As water trickles down, the lumps of alum are dissolved and a saturated alum solution is discharged at the base. A minimum tower height of 3 meters is required to ensure a saturated solution. This design requires no mechanical equipment other than weighing scales and flow-meters. The water treatment plant serving Santiago, Chile has successfully operated a 172,800 m^3/day plant using wooden saturation towers for alum feeding since 1959 (Poblete, 1964). Similar feeders have also been used in smaller plants.

A saturation tower for feeding lime solutions which has minimized some of the maintenance problems associated with conventional lime-suspension feeders (e.g. clogging pipes and orifices) has been used in Brazil. Although only slightly soluble, a lime solution with concentrations of 1200 to 1300 mg/liter can be maintained, if cold water is used. Lime is also more soluble if the lime particles are small. Figure 4-23 illustrates the lime saturator. Cold water flows from a constant level box, is metered, and then carried to a conical tank where it is saturated with lime. The saturated solution is removed by a collection trough located along the upper perimeter of the tank, and fed directly to the point of application. The inert material and deposits of calcium carbonate are extracted through a

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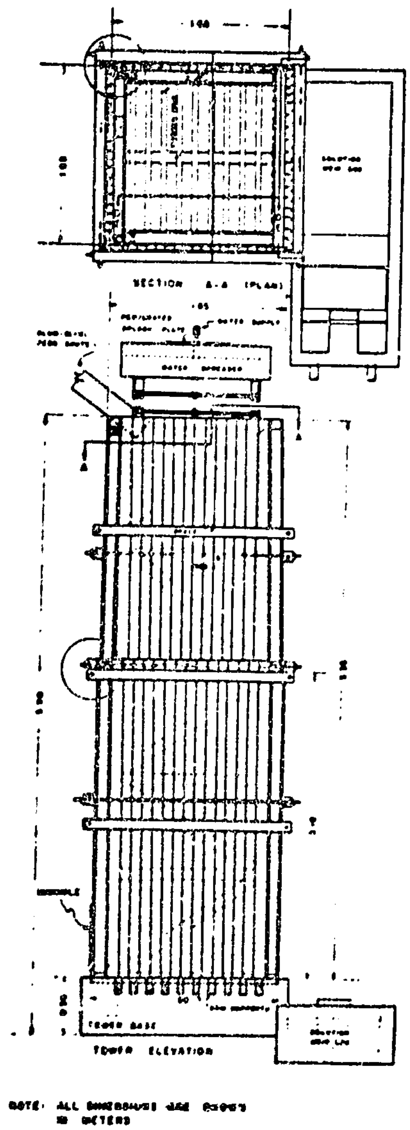
FIGURE 4-22

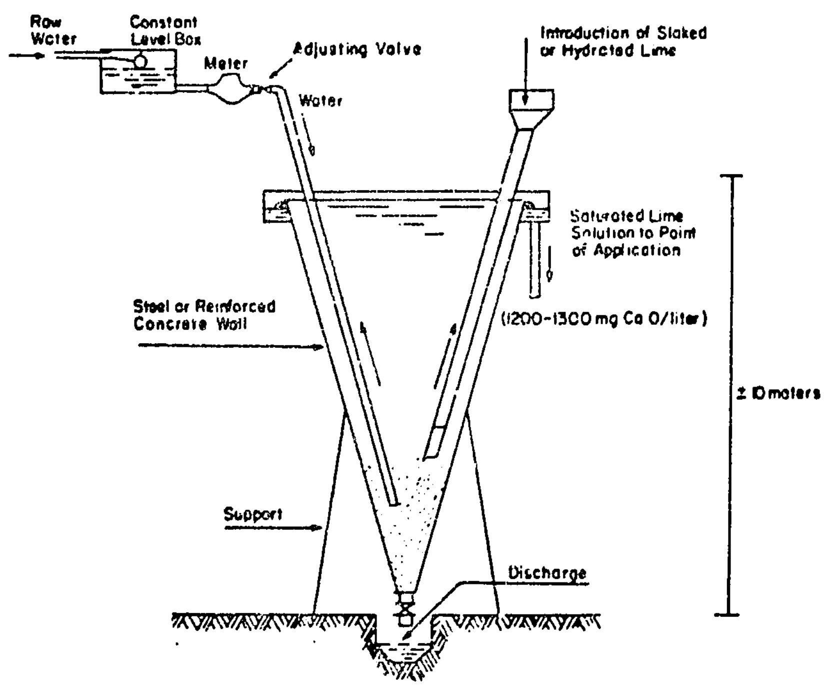
Wooden Saturation Tower for Alum Feeding
(developed in Chile)



[SOURCE: AID UNC/IPSED, 1967]

FIGURE 4-23

Lime Saturation Tower
(developed in Brazil)



[SOURCE: IRC, 1977a, p. 334]

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bottom drain. The tank is constructed generally from steel which is not corroded by lime, and therefore needs no protective coating.

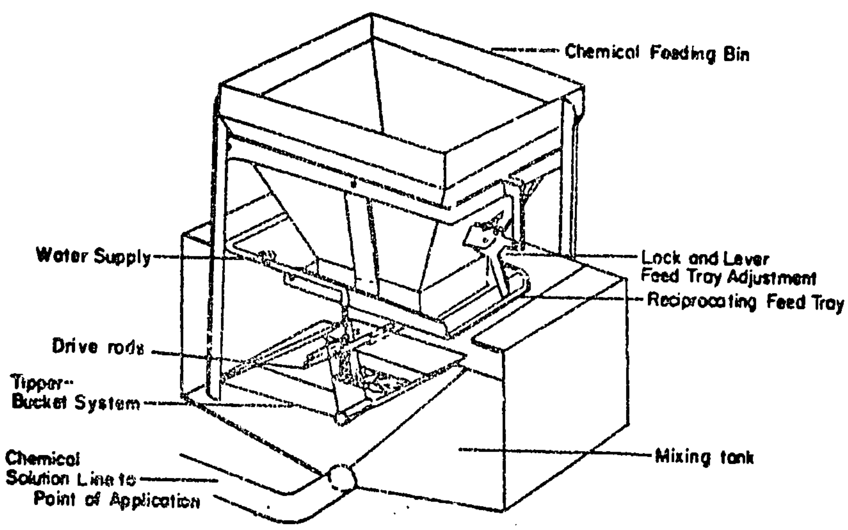
Dry-Chemical Feeders

Dry-chemical feeders consist generally of conical hoppers fitted at the bottom with either a volumetrically or gravimetrically operated device for the dispensing of the dry chemicals. Volumetric devices plow, push, or shake the chemical into a receiving chamber, where it is mixed with water, and conveyed to the point of application. Dry-chemical feeders enjoy some advantages over solution-type feeders, including (1) more accurate feeding; (2) longer unattended operation; and (3) eliminating the need for making up solutions or slurries. However, they are generally more complex and more difficult to maintain than solution-type feeders, particularly in humid climates where unprotected metal parts corrode easily and hygroscopic chemicals (e.g. alum) can clog the inside of hoppers or jam the devices that control the chemical dosage.

An unusual dry feeder, which does not require electrical power, was developed by the Moore Fluid Equipment Company of South Africa and is illustrated in Figure 4-24. It operates on the tipping bucket principle utilizing water as the drive. The water operates a tipping bucket which in turn reciprocates a stainless steel tray below a feed hopper causing the chemical to be discharged and to fall into a chamber

FIGURE 4-24

Hydraulically Operated Dry-Chemical Feeder
(developed in South Africa)



[SOURCE: adapted from Moore Fluid Equipment brochure, South Africa] beneath. There the water from the tipping bucket is discharged, creating turbulence and forming a solution or slurry of the chemical being fed, which is then fed by gravity to the point of application. Two methods are provided for adjusting chemical dosage: controlling the flow of the operating water, and adjusting the position of the tray below the feed bin which is effected by a lever and lock system. The unit also incorporates a bin agitator, consisting of a rubber hammer which strikes the chemical storage bin with every stroke of the feeder. The feeding range for this unit varies between 1 to 30 lbs per hour of hydrated lime or 2 to 60 lbs per hour of granulated aluminum sulphate. Some special features of this feeder are summarized here:

1. The drive water is not wasted but used for making the solution.

2. The feeder starts and stops automatically with the commencement and cessation of waterflow, respectively.

3. The feeder provides automatic proportionate feeding when the drive water is arranged as a percentage of total flow (e.g., an orifice is placed in the raw water delivery line in conjunction with an upstream bypass to drive the chemical feeder).

4. The feeder has an built-in bin agitator to prevent the arching and hold-up of stored chemicals.

5. The dual regulation of chemical feed (drive water flow regulation and tray height) is possible.

6) The bucket-tipper drive principle also mixes and flushes chemicals.

7) A wide feed range is available and can be adjusted while in operation.

V. HYDRAULIC RAPID MIXING

The function of a rapid mix system is to disperse the coagulant uniformly throughout the entire mass of water to ensure effective coagulation. This process is normally followed by a period of flocculation during which the water is gently mixed to promote agglomeration of the coagulated particles. Rapid mix units are located at the head end of the plant and are designed to generate intense turbulence in the raw water by either mechanical or hydraulic means. Rapid diffusion is necessary as the coagulation process, which comprises the hydrolysis of the coagulant and destabilization of the colloidal material, is completed almost instantaneously (less than one second). Inasmuch as hydraulic rapid mixers are capable of achieving high velocity gradients for rapid diffusion of coagulants without using mechanical equipment, they are preferred in developing countries. Moreover, they require no imported equipment, and are easily constructed, operated, and maintained with local materials and personnel.

This chapter examines several types of hydraulic rapid mixers that are designed for use in either open channels or pressure conduits. More space is devoted to open channel mixers since these are generally simpler and less costly, and above-water coagulant diffusers are readily accessible for cleaning.

Design Criteria

Two major criteria control the processes of rapid mixing and flocculation: intensity of agitation and the duration of agitation. They are defined for the design of such mixing processes by the velocity gradient (G) and detention time (t = Q/V). The velocity gradient for rapid mixers is determined from the following equations developed originally by Camp and Stein (1943):

$$G = (Qpgh_1/uV)^{1/2} \tag{5-1}$$

for hydraulic mixing

$$G = (P/UV)^{1/2} \tag{5-2}$$

for mechanical rapid mixing

where

P = power, $Qpgh_1$ in baffled channels, (watts, kg x m^2/sec^3)

p = density of the water (kg/m^3)

h_1 = head loss (m)

Q = flow (m^3/sec)

V = volume of the unit (m^3)

g = gravitational constant (9.82 m/sec^2)

t = detention time (sec)

G = velocity gradient (sec^{-1})

u = dynamic viscosity of water (poises, kg/msec)

Values of the density (p) and dynamic viscosity (u) for water of different temperatures are listed in Table 5-1.

TABLE 5-1: Variations of the Specific Gravity (Density) and Viscosity of Water with Temperature

TEMPERATURE, °C	0	5	10	15	20	25	30
Density, (p) (kg/m^3)	999.9	1000	999.7	999.1	998.2	997.1	995.7
Dynamic Viscosity, poises (u)x10^3 (kg/msec)	1.79	1.52	1.31	1.14	1.01	0.89	0.80

[SOURCE: adapted from Fair, Geyer, and Okun, 1968]

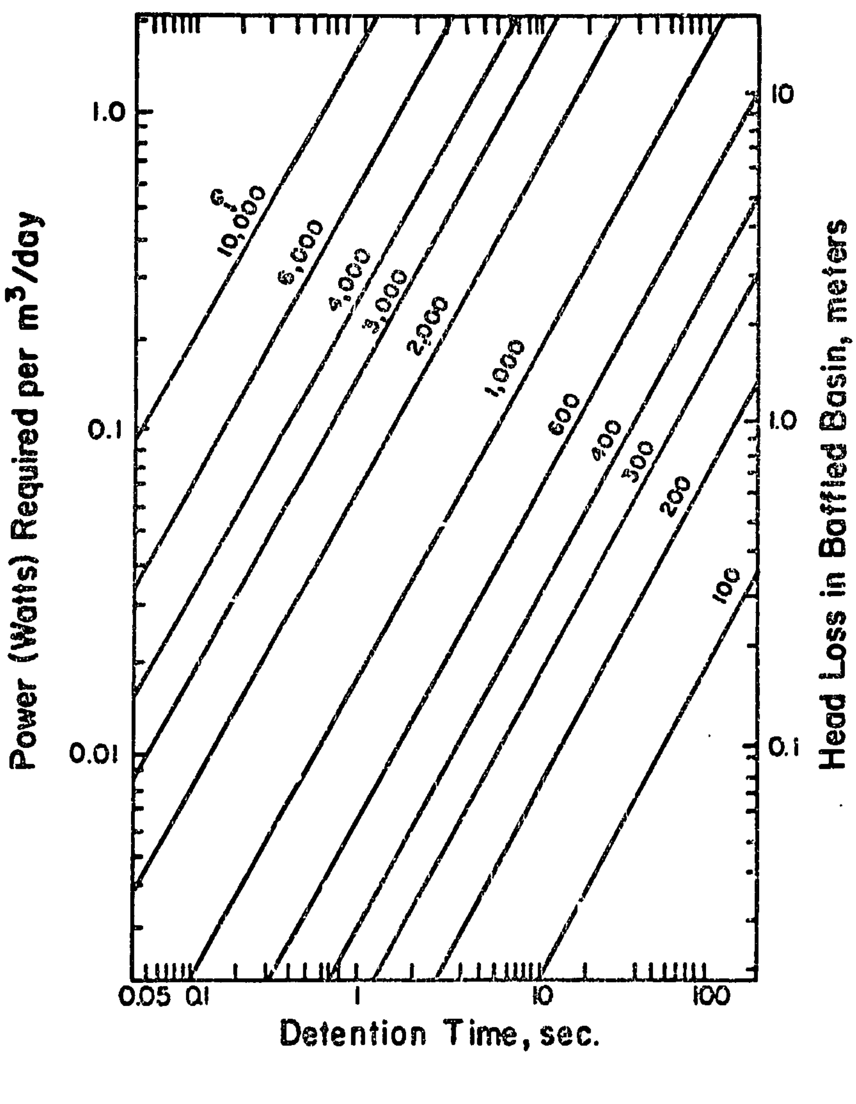
The head loss (h_1) in hydraulic mixers results from the turbulence created by the design; and from equations 5-1 and 5-2, is a measure of the power that is dissipated.

At present, no clear cut design criteria exist that prescribe appropriate G-values and detention times for the design of rapid mix units. However, the following general guidelines are suitable for open channel mixers (Hudson, personal communication): G-values of 500 sec^{-1} to 1000 sec^{-1} with 1 to 60 seconds detention time. The actual values that are obtained in practice vary substantially depending on the type of rapid mix unit employed.

The graph of Figure 5-1 is intended for rapid mix design and is based on equations 5-1 and 5-2. It is also useful for comparing the power required for mechanical mixing against the power required for the additional head for hydraulic mixing for achieving the same G-value. For example, a detention time of 2 seconds and velocity gradient of 1000 sec^{-1} would require either a head loss of about 0.3 meters per m^3/day for a hydraulic mixer or a power input of 0.03 watts per m^3/day for a mechanical mixer. The gravity head is normally acquired from raw water pumping, if the head is insufficient or unavailable in the raw water transmission main.

FIGURE 5-1

Power (head) Required for Rapid Mixing at 4°C



[SOURCE: adapted from Hudson, 1981, p. 68]

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Rapid Mixing Devices

The primary difference between mechanical and hydraulic rapid mixing is the manner by which they impart turbulence in the incoming raw water. For mechanical rapid mixers, the degree of turbulence is a function of the equipment's horsepower and is largely independent of flow; whereas the degree of turbulence for hydraulic mixers is measured by the loss in head and is dependent on flow. Mechanical mixers are generally proprietary devices whose major technical advantage is the flexibility they provide for adjusting the degree of turbulence to suit particular treatment needs. However, this advantage is of little consequence in places where skilled operators are unavailable to make such adjustments properly.

Hydraulic rapid mixers are designed for either of two types of flow conditions: viz., open channel flow or pressure flow in pipes. When feasible, open channel flow in concrete gravity channels is preferred, as such designs eliminate costly pipes and fittings, and can reduce the total capital cost of the plant. Moreover, rapid mixers in open channels are relatively simple, and have their component parts exposed and accessible for easy operation and maintenance. The general types of open channel hydraulic mixers described in this chapter are (1) hydraulic jump mixers; (2) flumes; and (3) weirs. Rapid mixers that utilize turbulent flow in pressure pipes and that are practical for developing countries are (1) hydraulic energy dissipators, and (2) turbulent pipe flow mixers.

Hydraulic Jump Mixers

This type of mixer includes a chute followed by a channel, with or without a drop in the elevation of the channel floor. The chute creates supercritical flow, the gently sloping channel provides a transition from supercritical to tranquil flow, which induces the jump, and the drop in the floor elevation defines the location of the jump. A diagram of a simple hydraulic jump mixer is shown in Figure 5-2.

The relative depths of the upstream and downstream water profile describe the basic conditions required for the formation of a hydraulic jump and can be calculated from the following equation (Fair, Geyer and Okun, 1968):

$$d_2/d_1 = 1/2[(1 + 8F^2)^{1/2} - 1] \qquad (5\text{-}3)$$

where

d_1 = depth of water upstream of the jump (m)

d_2 = depth of water downstream of the jump (m)

F (Froude's Number) = $[v_1/(gd_1)^{1/2}]^2$; where

v_1 = velocity of flow upstream of jump (m/s);

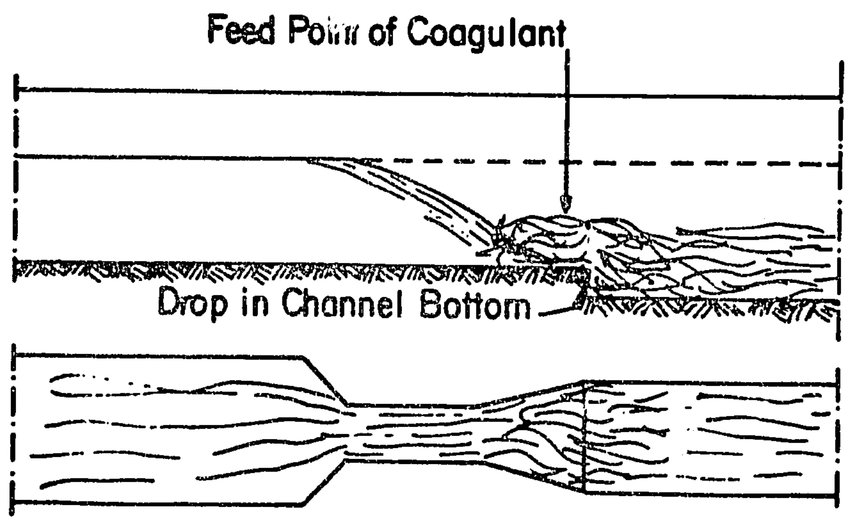
g = gravitational constant (9.81 m/sec^2)

A hydraulic jump is formed when the depth ratio (d_2/d_1) is greater than 2.4, the Froude number (F) then being greater than 2. When the Froude number is between 4 and 9, the energy consumed in turbulence can be between 45% and 70%,

\4.6

FIGURE 5-2

Hydraulic Jump Mixer



[SOURCE: adapted from IRC, 1981b, p. 219]

which is quite adequate for rapid mixing (Arboleda, 1973). Typical head losses are 0.3 meters or greater.

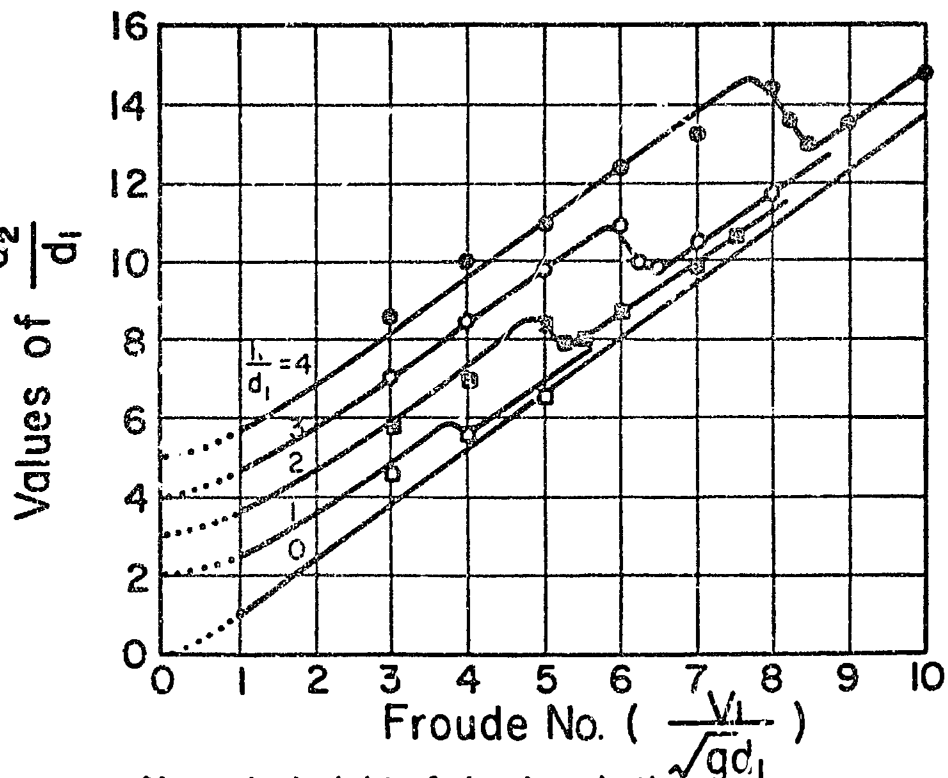
The elevation of the channel floor must be dropped to assure the location of the hydraulic jump. The drop is generally placed at the end of the expansion of a supercritical flow (see Figure 5-2). The curves shown in Figure 5-3 may be used for design purposes to determine the relative height of the drop required to stabilize a jump for any given combination of discharge, upstream depth, and downstream depth.

Parshall Flume

The Parshall flume, employed conventionally in water and wastewater treatment plants as a flow measurement device, is also effective as a rapid mixer when a hydraulic jump is incorporated immediately downstream of the flume. Advantages of Parshall flumes over other types of rapid mixers are: (1) the hydraulic jump obviates the need for mechanical agitation and minimizes clogging from suspended material in the water that would otherwise accumulate on the floor of the flume; (2) the Parshall flume can be used to measure flows; (3) the Parshall flume operates as a single head device with a minimum loss of head (about 1/4 of that required by a weir under similar flow conditions); and (4) the Parshall flume can be made entirely of materials available locally (e.g. concrete, wood).

FIGURE 5-3

Experimental Relations Among Froude Number (F), d_2/d_1 and h/d_1 for Hydraulic Jumps with an Abrupt Drop



Note: h = height of the drop in the channel floor
d_2 = depth of water downstream of the jump
d_1 = depth of water upstream of the jump

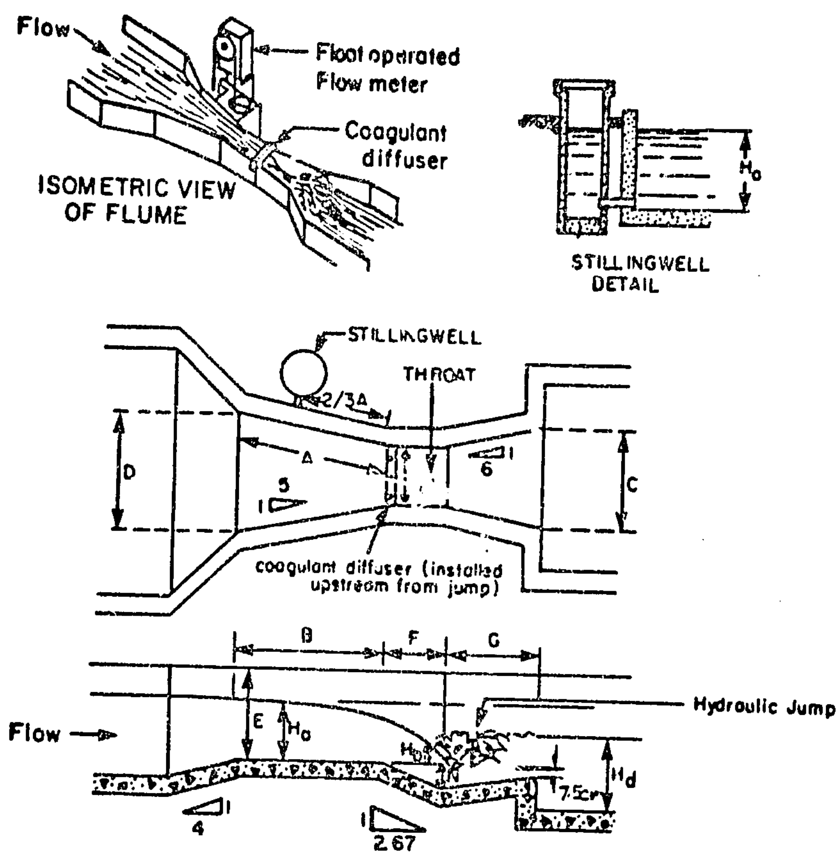
[SOURCE: Hsu, 1950, p. 991]

The Parshall flume consists of three principal sections: 1) a converging or contracting section at the upstream end leading to 2) a constricted section or throat, and 3) a diverging or expanding section downstream. The floor of the converging section is horizontal, the floor of the throat inclines downward, and the floor of the diverging section slopes upward (Figure 5-4).

The Parshall flume can be constructed in a wide range of sizes to handle virtually any flow range that is likely to be encountered in a water treatment plant. The width of the throat (W) is used to designate the size of a flume. Table 5-2 lists standard flume dimensions for various throat widths, designated by letters which appear in Figure 5-4; as well as the range of discharges corresponding to each flume size.

FIGURE 5-4

Parshall Flume Rapid Mixer



[SOURCE: adapted from Okun and Ponghis, 1975, p. 52]

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TABLE 5-2: Dimensions and Capacities of the Parshall Flume for Various Throat Widths[a]

THROAT WIDTH W mm	A mm	B mm	V mm	D mm	E mm	F mm	G mm	Q m^3/day	FREE-FLOW CAPACITY
152	620	600	390	400	600	300	600	122	9,550
304	1370	1340	600	850	900	600	900	274	39,500
456	1450	1420	750	1030	900	600	900	367	60,200
608	1530	1500	900	1210	900	600	900	1030	81,100
910	1680	1650	1200	1570	900	600	900	1500	123,000
1220	1830	1790	1520	1940	900	600	910	3190	166,000
1520	1680	1940	1830	2150	900	600	910	3920	210,000

[a]For the significance of the various letters, see Figure 5-4.

[SOURCE: adapted from Okun and Ponghis, 1975, p. 53]

It is important to maintain free-flow conditions in the Parshall flume if it is also to be used for flow measurement. This is defined as the condition under which the rate of discharge for any flume is dependent solely on the depth of water at the gauge point H_A in the converging section. The antithesis of free-flow is submerged flow, where the elevation of the water surface downstream from the flume is high enough to retard the rate of discharge; a condition that wastes energy and which should be avoided. Nevertheless, it is possible for Parshall flumes to withstand a high degree of submergence without significantly reducing the indicated rate of free-flow. It is such partially submerged flow which permits Parshall flumes to serve as effective rapid mix units in water treatment. A partially submerged flow is shown in Figure 5-4 where the backwater raises the downstream water surface, forming a hydraulic jump just downstream from the end of the throat.

The degree of submergence is often defined by the ratio of the two measured heads, H_b/H_A, obtained from the water levels in the throat (H_b) and upstream stilling well (H_A). In practice, however, it is very difficult to determine the value of H_b beforehand, for the purpose of calculating the submergence ratio. However, it has been shown experimentally that when submergence commences, the water surface levels in the downstream channel (H_d) and at the throat (H_b) are about the same. Consequently, the downstream level may be used for design purposes. The submergence ratio must be within the following limits in order to maintain free-flow conditions in the flume:

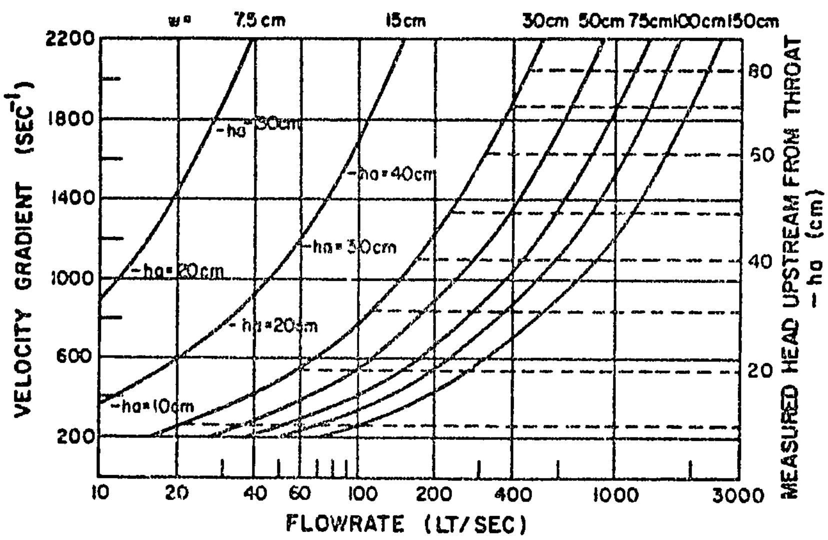
WIDTH OF THROAT (W)	MAXIMUM SUBMERGENCE (H_b/H_A or H_d/H_A) RATIO
<0.3 m	0.60
0.3 m $< W < 2.5$ m	0.70

An abrupt drop in the elevation of the channel floor immediately downstream of the flume is necessary to stabilize a hydraulic jump. The magnitude of the drop can be determined from the graph in Figure 5-3. Velocity gradients and measured upstream heads (H_A) for any combination of flow rates and standard throat sizes (W) in Parshall flumes may be determined from the graph in Figure 5-5, which is intended for design purposes. In general, velocity gradients of 500 sec^{-1} to 1000 sec^{-1} with detention times of 1 to 60 seconds can be used as design guidelines for Parshall flumes.

Parshall flumes may be built from concrete, wood, sheet metal, or plastic. Large flumes are usually constructed on site, but small flumes may be obtained as prefabricated structures to be installed in one piece. A series of Parshall flume rapid mixers for the Guandu plant in Rio de Janeiro, Brazil is shown in Figure 5-16.

FIGURE 5-5

Velocity Gradients for Different Flowrates in Parshall Flume Rapid Mixers



[SOURCE: adapted from Arboleda, 1973, p. 122]

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PALMER-BOWLUS FLUME

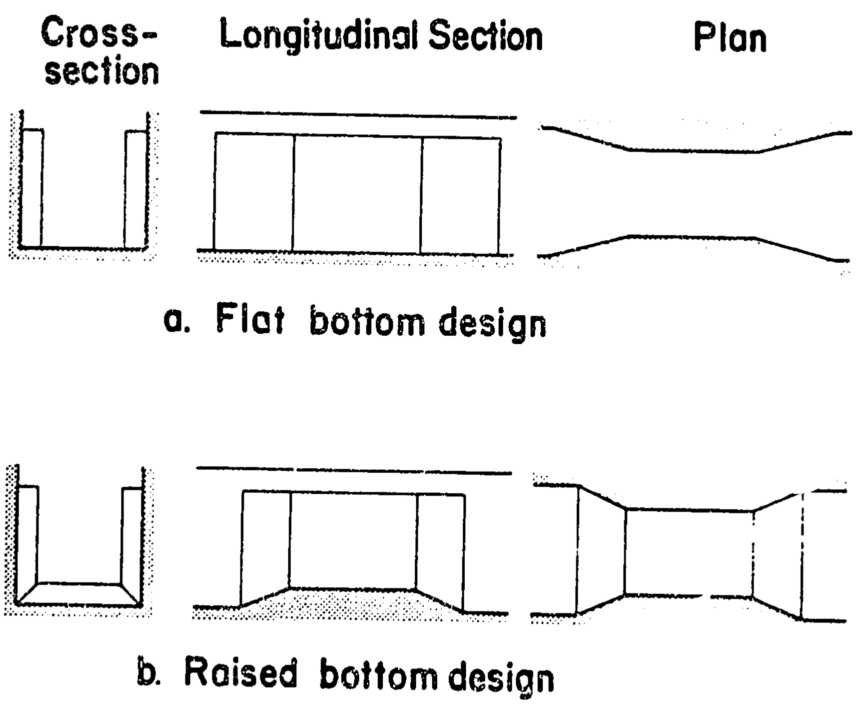
A simple modification of the Parshall flume is the Palmer-Bowlus flume, which is similarly formed by constricting the flow in an open channel or pipe. A principal advantage of such a flume is the comparative ease with which it can be installed in existing conduits, as it does not require a drop in the conduit invert, as would be required with a Parshall flume. Figure 5-6 shows two cross-sectional shapes of Palmer-Bowlus flumes installed in open channels; the length of the throat for each type is about equal to the average depth. Figure 5-7 shows the location of the hydraulic jump and preferred head measuring point. When installing Palmer-Bowlus flume, an adequate channel slope (which applies only to the downstream section) is necessary to maintain critical flow through the flume and prevent submergence. Such conditions are assured as long as the downstream depth of flow is not greater than 85% of the upstream depth. For new installations, a slight drop in the channel floor at the downstream side of the flume will assure free flow and stabilize the jump. In practice, Palmer-Bowlus flumes are not used as widely as Parshall flumes; hence information on their effectiveness in the field as rapid mixers could not be ascertained.

<u>Weirs</u>

Flow-measuring weirs are simple, but effective, methods of rapid mixing for plants having relatively small

FIGURE 5-6

Cross-sectional Shapes of Palmer-Bowlus Flumes



[SOURCE: Grant, 1979, p. 43]

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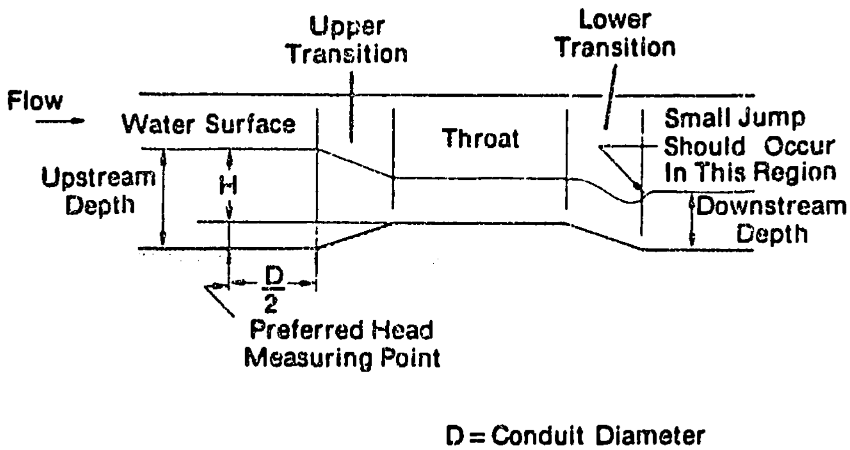
FIGURE 5-7

Free-flowing Palmer-Bowlus Flume



[SOURCE: Grant, 1979, p. 46]

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capacities. A weir is low in cost, relatively easy to install, and can also be used as a flow measuring device. However, a weir normally operates with a rather significant loss of head (about 0.3 to 0.6 m) and must be periodically cleaned to prevent deposition of sediments on the upstream side of the weir. Weirs are generally less expensive to fabricate and install than flumes, particularly Parshall flumes, due to simpler design and the types of materials required. Weirs are constructed by placing a thin metal plate (3 mm to 7 mm thick) or concrete wall across the flow and forcing the flow through a specified opening. This opening may be of several configurations, as shown in Figure 5-8.

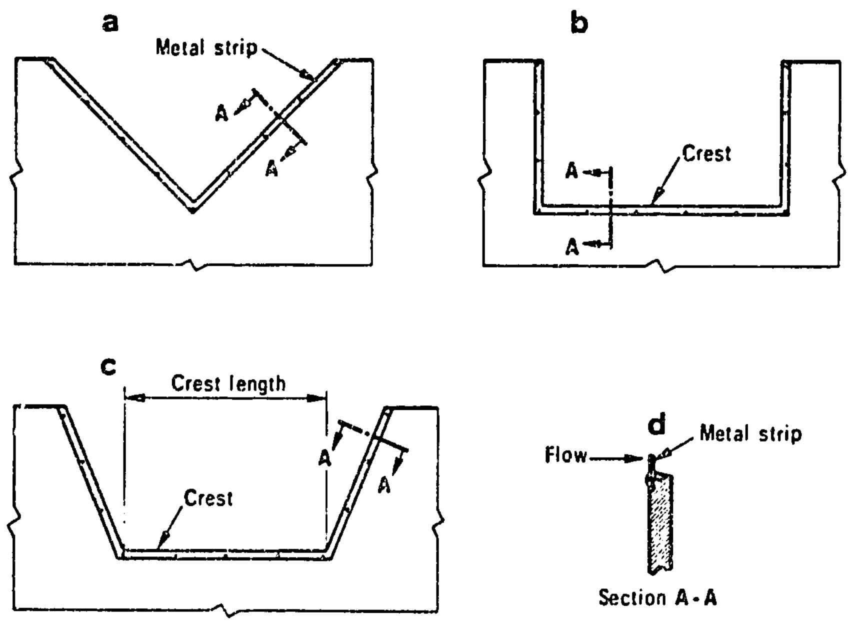
Triangular weirs are normally used for low flows, whereas rectangular weirs are used for larger flows. Because they distribute the flow more uniformly across the channel width, rectangular weirs are preferred when using above-water coagulant diffusers. Minimum and maximum flow rates for both types of weirs are listed in Tables 5-3 and 5-4. Weirs can be made of wood with steel edges (Figure 5-8), plastic, or asbestos-cement.

The sudden drop in the hydraulic level over the weir induces the turbulence in the water for rapid mixing, and chemicals are added at this "plunge" point with the help of a diffuser. The vertical fall of the raw water over the weir should be at least 0.10 meters in order to obtain a

FIGURE 5-8

Measuring Weirs:

a) V-Notch; b) Rectangular; c) Trapezoidal; d)Sections A-A



[SOURCE: Okun and Ponghis, 1975, p. 50]

TABLE 5-3: Min. and Max. Recommended Flow Rates for V-Notch Weirs

V-NOTCH ANGLE	MIN. HEAD		MIN. FLOW RATE		MAX. HEAD		MAX. FLOW RATE	
	ft	cm	MGD	m^3/day	ft	cm	MGD	m^3/day
22-1/2°	0.2	6.0	.006	22.7	2.0	61.0	1.82	6900
30°	0.2	6.0	.008	30.3	2.0	61.0	2.47	9360
45°	0.2	6.0	.012	45.5	2.0	61.0	3.78	14,300
60°	0.2	6.0	.017	64.4	2.0	61.0	5.28	20,000
90°	0.2	6.0	.029	109	2.0	61.0	9.14	34,600

[SOURCE: adapted from Grant, 1979, p. 21]

TABLE 5-4: Min. and Max. Recommended Flow Rates for Rectangular Weirs with End Contractions

CREST LENGTH		MIN. HEAD		MIN. FLOW RATE		MAX. HEAD		MAX. FLOW RATE	
ft	cm	ft	cm	MGD	m^3/day	ft	cm	MGD	m^3/day
1	30.5	0.2	6.0	.185	700	0.5	15.2	.685	2,590
1.5	45.7	0.2	6.0	.281	1060	0.75	22.9	1.89	7,160
2	61.0	0.2	6.0	.377	1430	1.0	30.5	3.87	14,700
2.5	76.2	0.2	6.0	.474	1800	1.25	38.1	6.77	25,600
3	91.4	0.2	6.0	.570	2160	1.5	45.7	10.7	40,500
4	122	0.2	6.0	.762	2890	2.0	61.0	21.9	83,000
5	152	0.2	6.0	.955	3620	2.5	76.2	38.3	145,000
6	183	0.2	6.0	1.15	4360	3.0	91.4	60.4	229,000
8	244	0.2	6.0	1.53	5800	4.0	122	124	470,000
10	305	0.2	6.0	1.92	7270	5.0	152	217	822,000

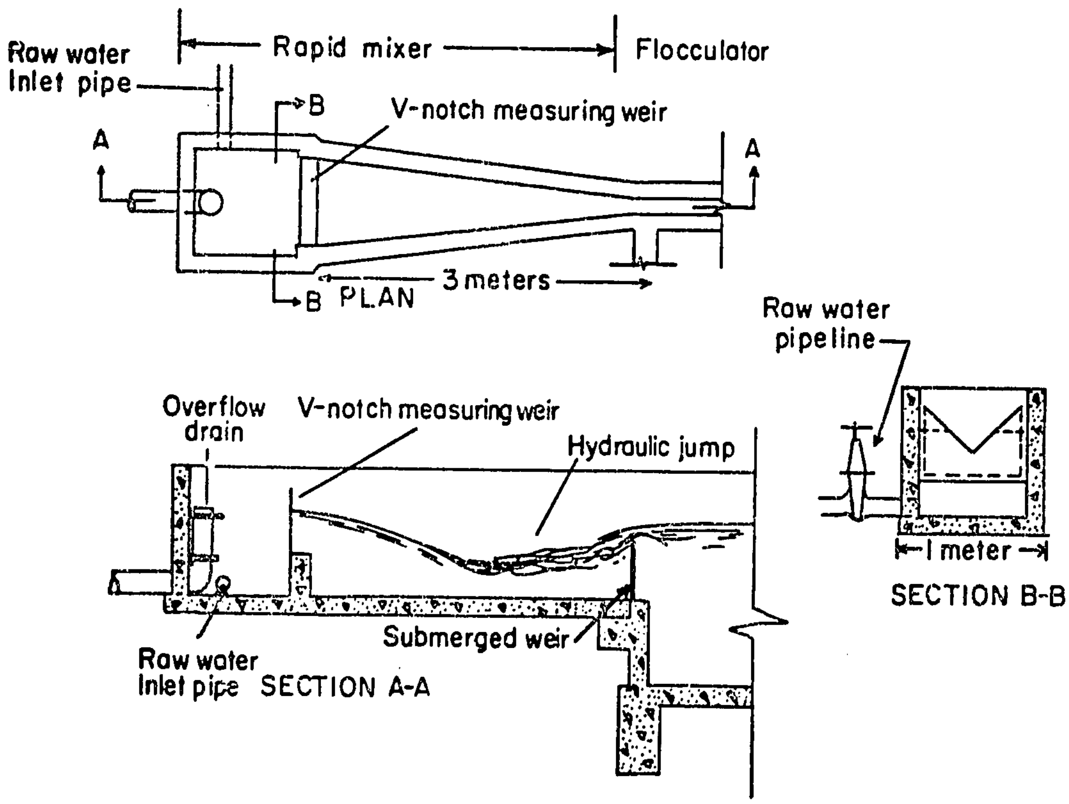
[SOURCE: adapted from Grant, 1979, p. 25]

G-value of about 1000 sec^{-1}. The height of the coagulant feeder over the weir should be at least 0.3 meters in order that the speed of the falling coagulant solution is high enough to penetrate the nappe thickness. To utilize the energy from the weir effectively, a small receiving chamber should be constructed below the weir where rapid mixing agitation can take place. A simple chamber consisting of a submerged weir 3 meters downstream from the V-notch weir and converging side walls, as depicted in Figure 5-9, is a suitable design. The submerged weir induces a hydraulic jump within the chamber for additional mixing. Another design that is employed in several plants in India incorporates a baffled channel, which immediately follows the measuring weir (Figure 5-10). Turbulence is induced initially by the fall of water over the V-notch weir and then continues in the baffled channel as the water is conveyed to the flocculation basin. The mixing channel is sloped at 1 to 50 and contains baffles turned at 45°.

For large treatment plants, the incoming flow at the head of the plant may be split equally among a series of weirs at the same elevation so as to limit the head loss over each weir. An interesting weir system for rapid mixing is used in a 250,000 m^3/day water treatment plant that serves the city of Nairobi, Kenya. In this plant raw water enters the weir chamber through a pressure conduit and discharges onto a concrete pedestal enclosed by

FIGURE 5-9

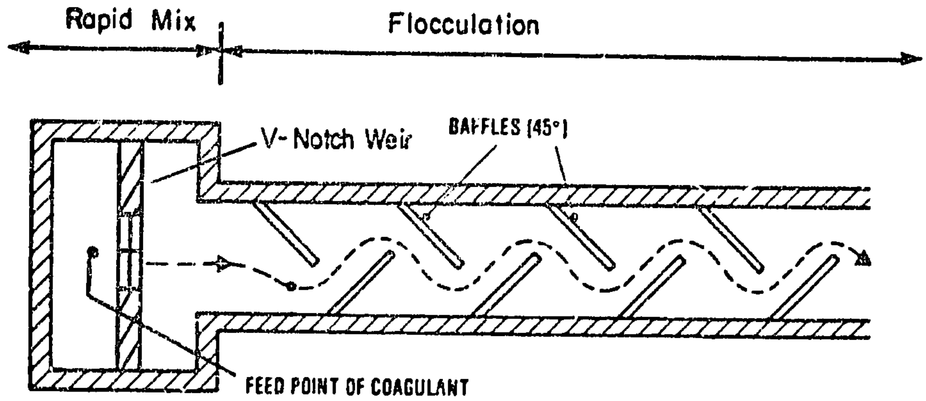
Weir Rapid Mixer for a Peruvian Treatment Plant
(3110 m^3/day)



[SOURCE: PAHO, personal communication]

FIGURE 5-10

V-notch Weir and Baffled Channel for Rapid Mixing; Plan View



[SOURCE: adapted from IRC, 1981b, p. 218]

sharp-crested weirs along its perimeter. The water flows radially outward, over the weir and into a receiving chamber one meter below the pedestal elevation. Turbulence in the form of standing rollers occupies this space with a retention time of about 2 to 5 seconds. Figure 5-11 shows the good mixing that is achieved with this type of design.

Baffled Mixing Chambers

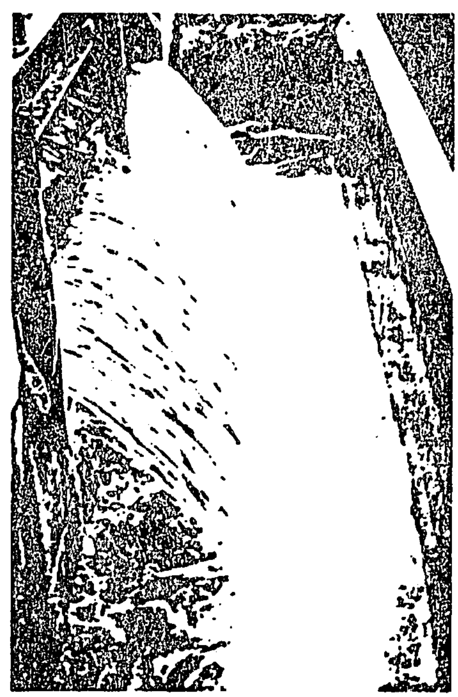
In general, baffled mixing chambers are not recommended for rapid mixing because of their plug-flow characteristics which are not conducive to turbulent mixing in short time periods. Their main contribution in water treatment lies in the flocculation stage, where gentler mixing over a longer period of time is desired (see Chapter 6). As mentioned earlier, baffled mixing chambers may be used in conjunction with weirs as a composite rapid mix unit for special configurations in small water treatment plants.

Hydraulic Energy Dissipators

In places that enjoy high residual hydraulic pressure at the plant headworks, a viable option is to use this residual pressure for rapid mixing. By installing hydraulic energy dissipators, such as stilling basins or jet orifices, turbulence is created as the water passes through their openings into a mixing chamber. Three configurations for such mixers are shown in Figures 5-12 and 5-13.

FIGURE 5-11

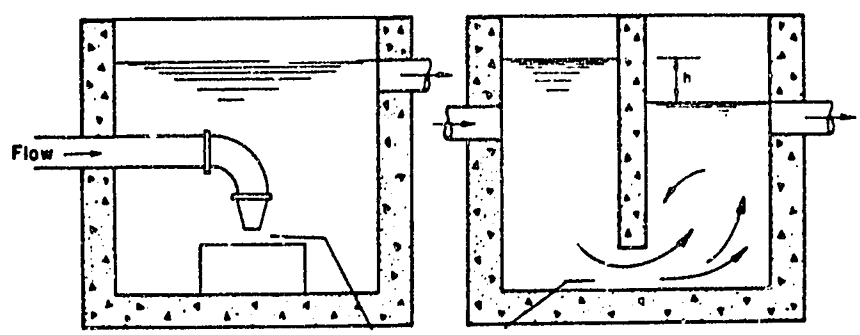
Weir Rapid Mixer for a Plant in Nairobi, Kenya



[SOURCE: Singer, personal communication]

FIGURE 5-12

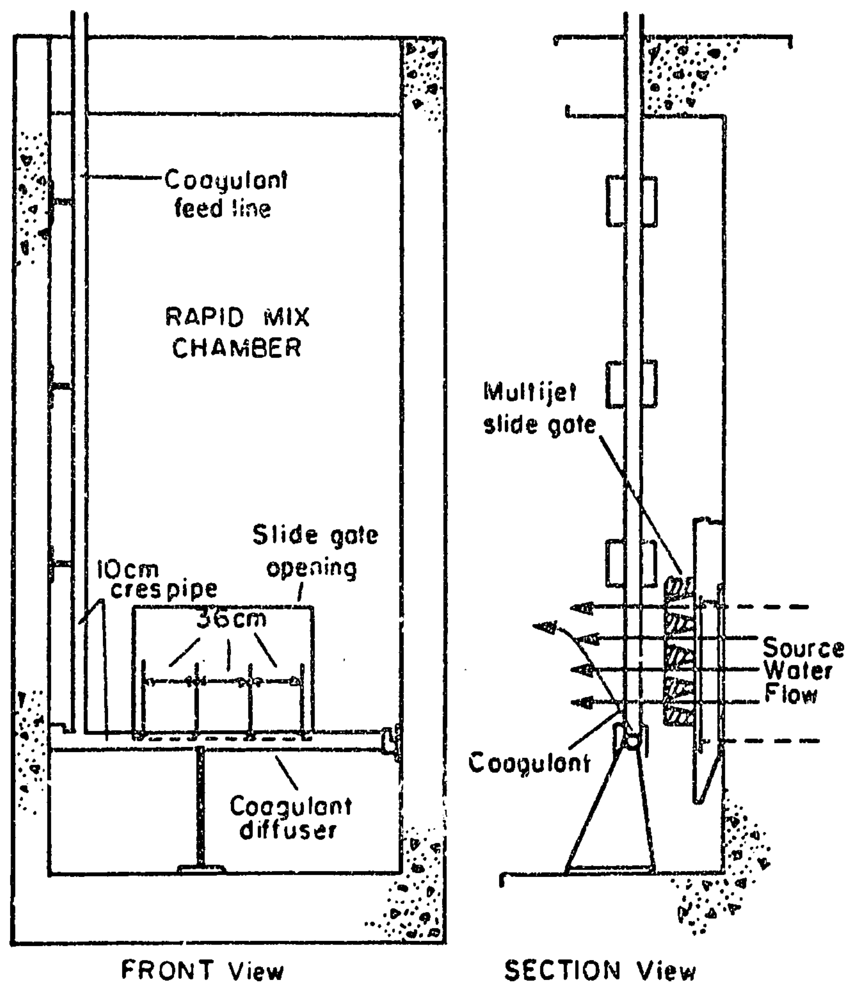
Two Types of Hydraulic Energy Dissipators for Rapid Mixing



[SOURCE: Arboleda, 1973, p. 115]

FIGURE 5-13

Multi-jet Slide Gate for Rapid Mixing at the Oceanside Plant - Arcadia, California



[SOURCE: MacDonald and Streicher, 1977, p. 88]

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TURBULENT PIPE FLOW MIXERS

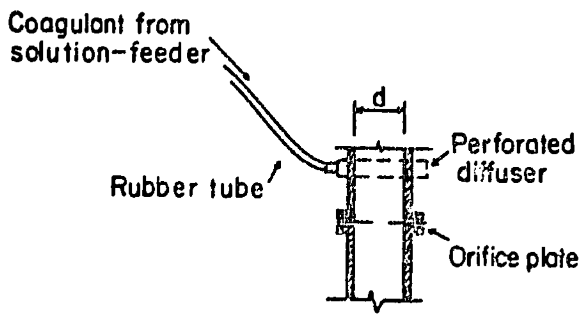
Several recently developed methods for diffusing chemicals in turbulent pipe flow have drawn considerable attention from researchers due to their practicality, simplicity, and relatively low cost (Chao and Stone, 1979). Installations of turbulent pipe flow mixers in a number of water treatment plants have been reported by Kawamura (1976). In the designs listed, G-values of 700 to 1000 sec^{-1} are attained in a mixing time of about one second.

Hydraulic mixing in pipes can be achieved in a variety of ways. In Brazil, a short length of pipe (0.3 to 0.7 m) is filled with glass balls or quartz pebbles, providing a velocity gradient for mixing greater than 800 sec^{-1} with a loss of head from 0.2 to 0.25 m. Modular plants designed by CEPIS (1982) employ an orifice plate placed in the raw-water pipe (see Figure 5-14). The orifice diameter is chosen so that it causes a loss of head that produces a velocity gradient of about 1000 sec^{-1} (the graph in Figure 5-1 may be used for determining G-values for given head losses). Pipes can also be fabricated with fixed, sloping baffles inside them to impart turbulence to the passing water.

Turbulent pipe flow mixers may present operation and maintenance problems in developing countries because (1) an auxiliary pump is normally required to inject the chemicals into the flow stream unless special arrangements are made

FIGURE 5-14

Orifice Plate for Rapid Mixing



[SOURCE, adapted from CEPIS, 1982, vol. 2, plan no. 26]

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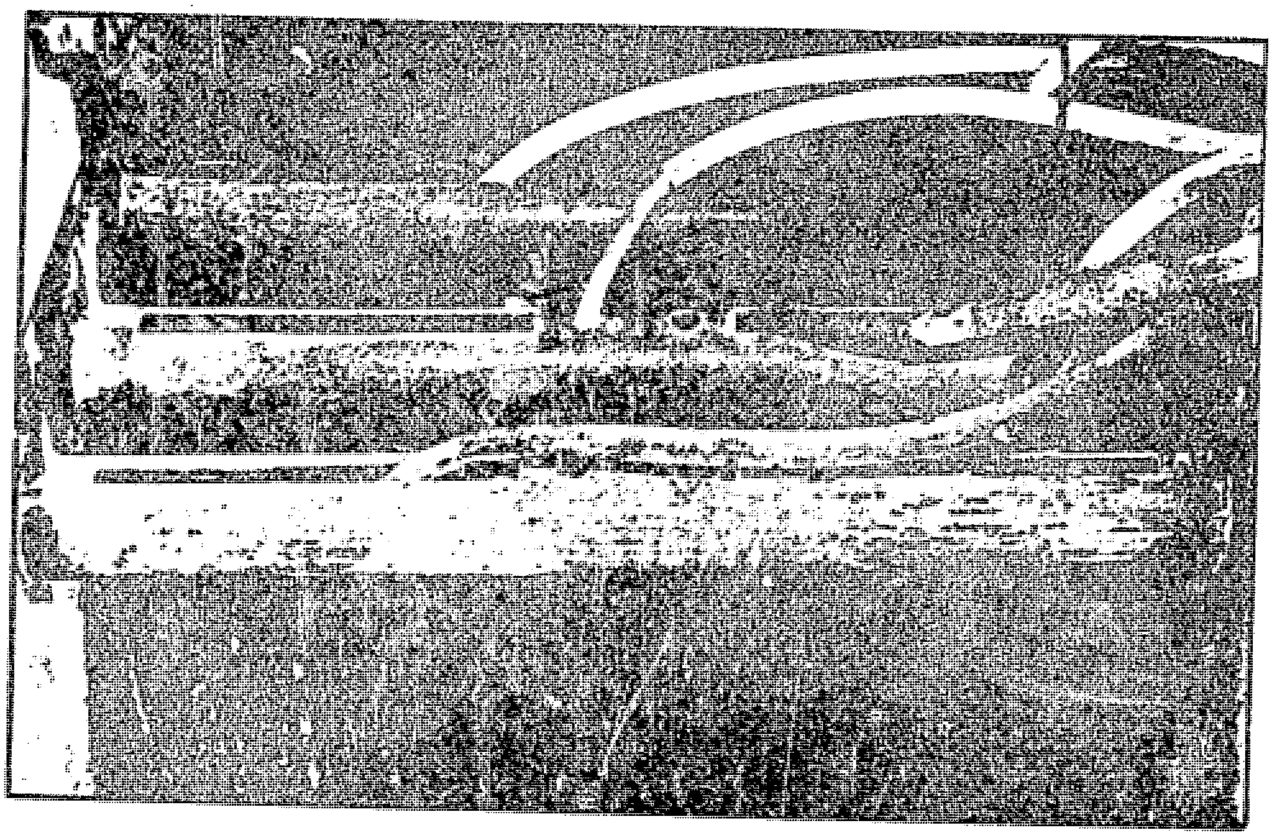
for providing enough head for gravity feed; and (2) the small feed openings in the diffuser tend to clog, hence the diffuser should be removable or exposed to allow for easy cleaning. However, they can be effective and economical rapid mixers if pumps are not required and designs permit easy maintenance.

Application of Coagulants in Open Channels

For open-channel mixers, the coagulant should be applied at a point immediately upstream of the zone of maximum turbulence by means of a perforated trough or perforated pipe diffuser. The pipe diffuser can be easily fabricated from a plastic pipe by drilling ports 0.6-1.3 cm in diameter and not more than 15 cm apart so as to evenly distribute the coagulant. In places where low-grade coagulants must be used, a trough fitted with triangular weirs on one side may be preferable to using perforated-pipe diffusers, since in the latter case, impurities in the coagulant are likely to clog the holes frequently. At least two sets of diffusers are desirable so that the coagulant can be fed continuously even when one diffuser is removed from operation for cleaning. Two photographs of perforated pipe diffuser systems for a V-notch weir and Parshall flume rapid mix chamber are shown in Figures 5-15 and 5-16, respectively. The diffusers are located above the plunge

FIGURE 5-15

Plastic-pipe Diffuser for a Weir Rapid Mixer in Malaysia



[SOURCE: Ching, 1979, p. 3]

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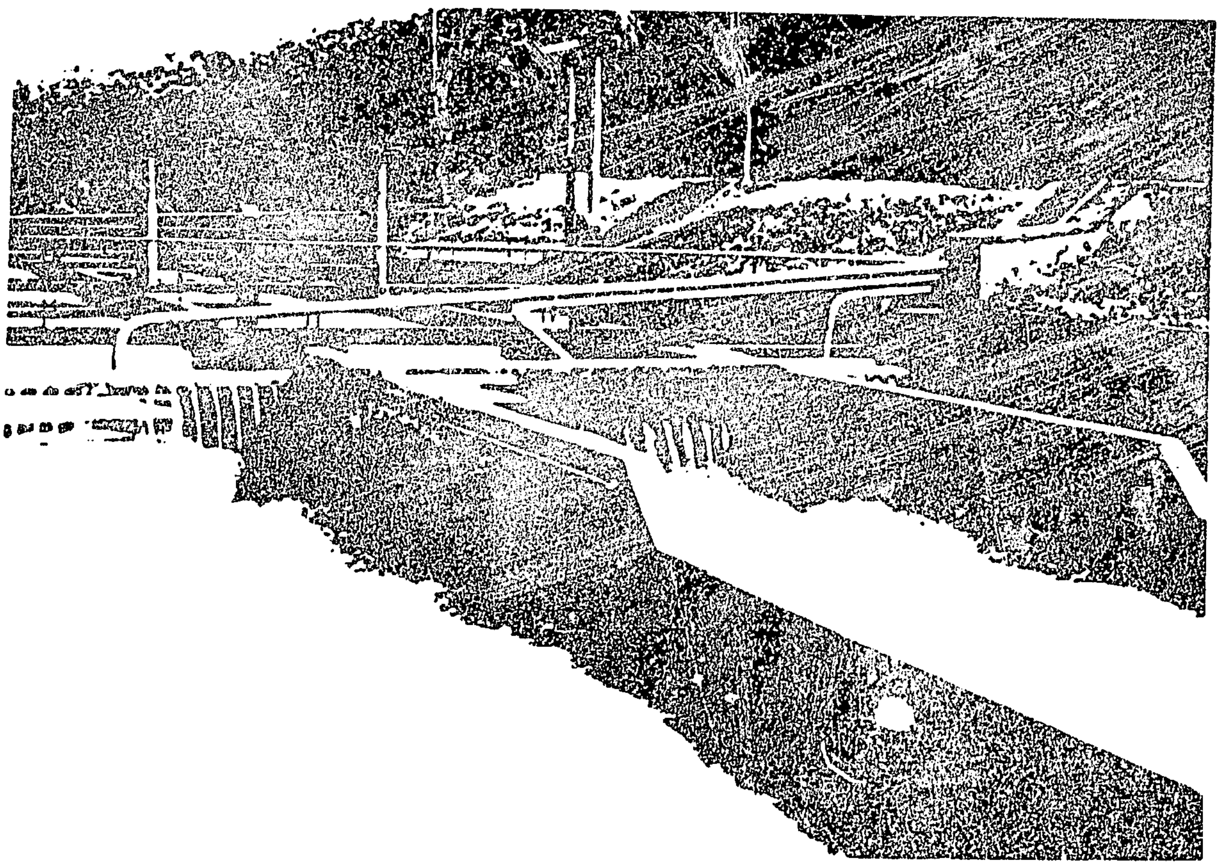
FIGURE 5-16

Parshall Flume Rapid Mixer in the Guandu Plant - Rio de Janeiro, Brazil



[SOURCE: Hudson, personal communication]

point for weir mixers, and directly upstream from the hydraulic jump for Parshall flumes.

Flow Measurement Systems in Open Channels

The flumes and weirs described in this chapter for use as hydraulic mixers can be readily adapted for flow measurement. The constrictions that are formed in the channel by flumes and weirs change the water level upstream from the constriction by a known function. Thus, the flow rate through an open channel can be derived by a measurement of this water level.

The following discharge equations are applicable for the given devices as long as free-flow conditions are maintained in the constriction (Grant, 1979):

90° V-notch weir	$Q = 1.38\ H_W^{5/2}$	(5-6)
rectangular weir	$Q = 1.84\ BH_W^{3/2}$	(5-7)
Parshall flume	$Q = 2.27\ WH_A^{3/2}$	(5-8)
Palmer-Bowlus flume	$Q = 1.66H_A^{3/2}W$	(5-9)

where

Q = discharge (m^3/s)

H_W = head on weir (m)

H_A = depth at entrance to the flume at specified measuring point (m)

B = depth at entrance of weir (m)

W = width of throat (m)

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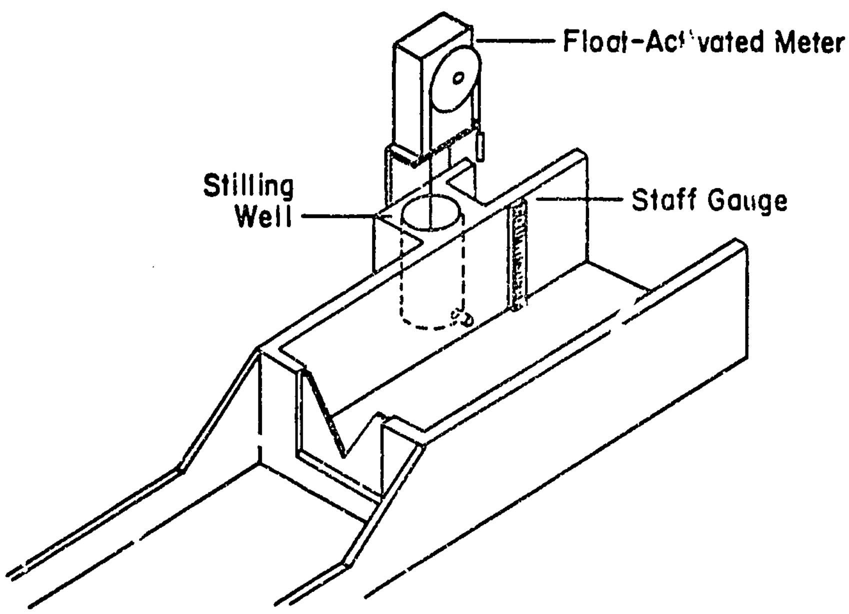
Measuring of open channel flow rates may be done simply using a stilling well and float-actuated recorder, an indicator or a staff gauge (see Figure 5-17). The stilling well suppresses any surges that are present in the water flowing through the channel due to wind action, waves, etc. The head connection line between the open channel and the stilling well should have a small cross-sectional area with respect to the stilling well. The float is usually conically shaped to provide stability. The wire or chain leading from the float is draped over a pulley behind the indicator. A counterweight is attached to the free end of the chain. As the float travels up with water level, and down with the aid of the counterweight, the flow is read manually and/or recorded. The stilling well should be large enough and provided with a drain to facilitate cleaning. For accurate measurement of rapidly fluctuating flows, smaller wells are necessary, so that the water level in the well adjusts quickly to changing flows in the channel.

A staff gauge can aid in the zero adjustment of the flow meter. A fixed scale is placed securely on one side of the open channel so that the water level in the channel can be read directly in order to calibrate the float-actuated meter. Calibration techniques for such installations are covered by Grant (1979). For convenience, the staff gauge or

FIGURE 5-17

Flow Measurement System Consisting of a Stilling Well, Flow-Activated Meter and Staff Gauge



[SOURCE: Grant, 1979, p. 79]

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the indicator can be calibrated by the engineer to read in flow units, so the conversion need not be made by the operator.

VI. HYDRAULIC FLOCCULATION

Flocculation is the process of gentle and continuous agitation during which suspended particles in the water coalesce into larger masses so that they may be removed from the water in subsequent treatment processes, particularly by sedimentation. The flocculation step is omitted in direct filtration plants. Flocculation follows directly after the rapid mix process, and like rapid mixing, the agitation may be induced either by mechanical or hydraulic means. Mechanical flocculators are preferred in the industrialized countries because of their low head loss and greater versatility, i.e. the speed of the mechanically-operated paddles can be adjusted to suit variations in flow, temperature, or raw water quality. Furthermore, mechanical flocculators are readily available from proprietors in those countries in a variety of designs to suit any mode of operation. The principal elements of mechanical flocculator systems are agitator impellers, drive motors, speed controllers and reducers, transmission systems, shafts, and bearings. The cost and added complexity of mechanical flocculator systems introduce additional complications, particularly in regards to operation and maintenance, and hence are not well suited for developing countries.

A more practical approach is to use hydraulic flocculators which do not require mechanical equipment, nor a continuous power supply, and which can be built primarily from concrete, brick, wood, or masonry with local labor at relatively low cost. Moreover, several hydraulic flocculation systems operate under plug-flow conditions (plug-flow, under ideal conditions, is achieved when water flows through a chamber at a uniform rate without intermixing) which minimize short-circuiting of the flow (i.e. when a portion of the incoming flow of water traverses the flocculation chamber in a much shorter time than the nominal detention period). Short-circuiting, an inherent problem of mechanical flocculators, is alleviated somewhat in practice by installing a series of successive compartments in the flocculation chamber.

The major shortcomings of hydraulic flocculators have been reported widely in the technical literature:

1) No flexibility to respond to changes in raw water quality.
2) The hydraulic, and consequent flocculation parameters, are a function of flow and cannot be adjusted independently.
3) The head loss is often appreciable.
4) Cleaning may be difficult.

These shortcomings are the reasons why hydraulic flocculators have not continued to be used extensively in the industrialized countries. Of 42 plants in the US built between 1908 and 1932, 30 had hydraulic flocculators (ASCE, 1940). However, it is possible to mitigate these shortcomings with properly designed systems that will function under a reasonably wide range of operating conditions. In fact, new designs for hydraulic flocculators and improvements in older designs have been implemented and are operating successfully in water treatment plants in Latin America (Azevedo-Netto, personal communication) and, interestingly enough, in several plants in California where high technology is readily available (MacDonald and Streicher, 1977).

This chapter examines several types of hydraulic flocculators that are appropriate for water treatment plants in developing countries. Baffled channel flocculators are the most widely used hydraulic method, particularly in Latin America. Gravel-bed flocculators have been installed during the last 10 years in several small water treatment plants in India (Kardile, 1981), and have been tested experimentally to ascertain their potential for use in Brazil (Richter and Moreira, 1981). Flocculators that use the jet action of the water to impart turbulence, such as the Alabama type and heliocoidal-flow type, have been used to a lesser extent, primarily because most engineers are unfamiliar with their design and operation. However, the Alabama-type flocculator has been used in small plants in Brazil. Staircase-type flocculators, developed recently in Brazil, and surface-contact flocculators, developed recently in India, are also examined in this chapter.

Design Criteria

The basic formulae for calculating velocity gradients (G) in flocculators are the same as those for rapid mixers, namely (Camp and Stein, 1943):

$$G = (Q\ p\ g\ h_1/uV)^{1/2} = (p\ g\ h_1/ut)^{1/2} \qquad (6\text{-}1)$$

for hydraulic flocculation

$$G = [P/(uV)]^{1/2} \qquad (6\text{-}2)$$

for mechanical flocculation

where

G = velocity gradient (sec^{-1})

p = density of water (kg/m^3)

h_1 = head loss (m)

u = dynamic viscosity (kg/msec)

t = detention time, Q/V (sec)

Q = flow (m^3/sec)

P = power, Qpgh (watts, kgm^2/sec^3)

V = volume of unit (m^3)\

g = gravitational constant (9.81 m/sec^2)

Values of the density (p) and dynamic viscosity (μ) for water of various temperatures are listed in Table 5-1. The above equations are the basis for the graph of Figure 6-1, which allows one to determine the velocity gradient (G) for a known head loss (h_1) and detention time (t) in

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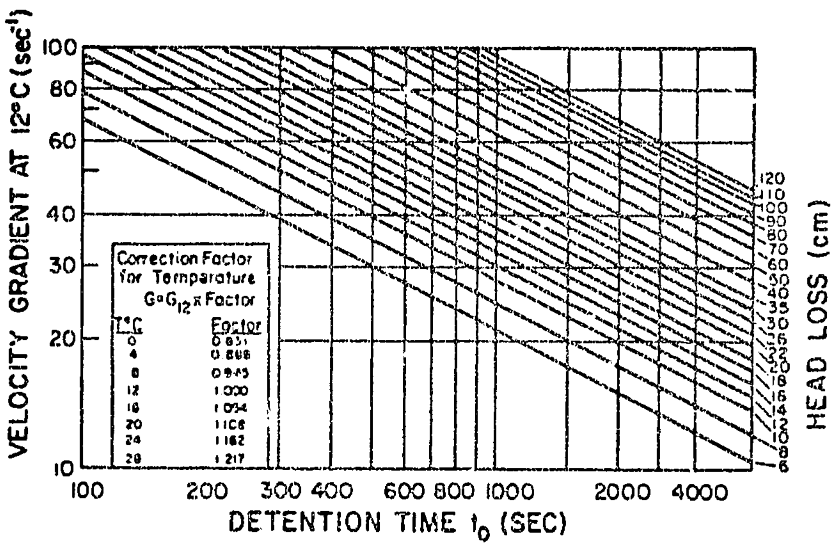
FIGURE 6-1

Velocity Gradients in Hydraulic Flocculators for Different Detention Times (t_0) and Head Losses (h_1) at a Temperature of 12°C



[SOURCE: Arboleda, 1973, p. 134]

hydraulic flocculators. The graph is calibrated for 12°C water temperature. Conversion factors for other water temperatures are listed in the table within the figure. For example, a hydraulic flocculator having a detention time of 15 minutes (900 seconds) and a head loss of 0.3 m would give a velocity gradient of about 55 sec^{-1} at 20°C. G-values for particular types of flocculator designs may also be obtained from formulae presented later in this chapter.

In the design of flocculation systems, the total number of particle collisions, and thus the floc formation action, is indicated as a function of the product of the velocity gradient and the detention time, Gt. The range of velocity gradient (G) and Gt values given in Table 6-1 have been shown in practice to be the most effective for plants using alum as the primary coagulant. Nonetheless, in order to obtain appropriate values for particular designs and water characteristics to provide for the optimal formation of flocs, laboratory jar testing or pilot plant studies should be conducted on the water to be treated.

Velocity gradients in a flocculation basin can be tapered to be high at the inlet end and low at the outlet end to achieve more efficient mixing and agglomeration of the floc particles. Such a design can reduce the magnitude of the shearing forces on the flocs as they agglomerate, and thereby reduce the chance of floc break-up. It is desirable to provide a tapered velocity gradient that is high at the

TABLE 6-1: Flocculator Design Criteria

Design Factor	G (sec^{-1})	t (sec)	Gt
Range	10 to 100	1200 to 1800	30,000 to 75,000
Typical Value	45 to 90	1800	50,000 to 70,000

[SOURCE: Hudson, personal communication]

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inlet side of the flocculator and low at the outlet side. Designs that yield tapered velocity gradients are discussed below.

Baffled Channel Flocculators

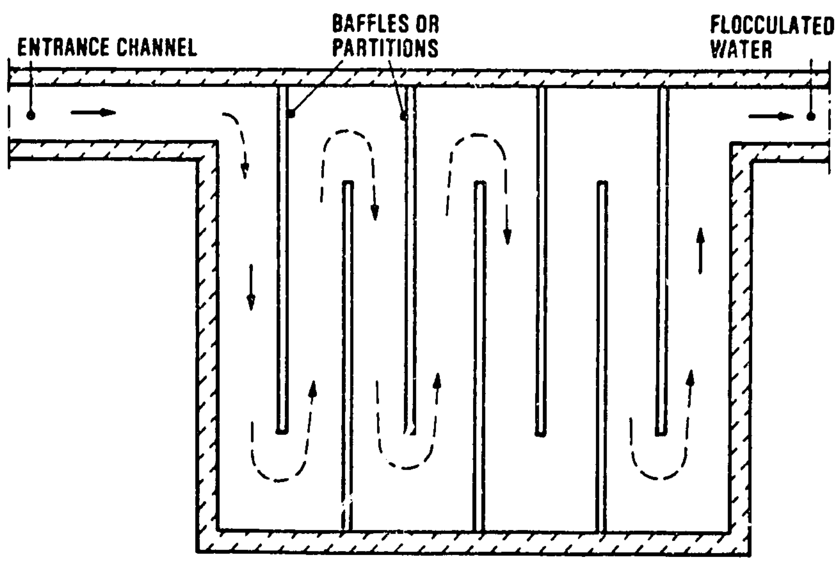
In baffled channel flocculation, mixing is accomplished by reversing the flow of water through channels formed by around-the-end or over-and-under baffles (Figures 6-2 and 6-3). A distinct advantage of baffled channel flocculators is that they operate under plug-flow conditions which free them from short-circuiting problems.

Horizontal-flow flocculators with around-the-end baffles are sometimes preferred over vertical-flow flocculators with over-and-under baffles because they are easier to drain and clean, and the head loss, which governs the degree of mixing, can be more easily changed by installing additional baffles or removing portions of existing ones. However, vertical-flow units have been used successfully in Brazil and in the US (see Figure 6-4) and are appropriate for specific applications, such as, for example, where a scarcity of land prohibits the use of larger horizontal-flow flocculators.

The water depth in the channels of vertical flow units can be as high as 3 meters, and therefore, less surface area is required than with horizontal units. The major problem with such flocculators is the accumulation of settled

FIGURE 6-2

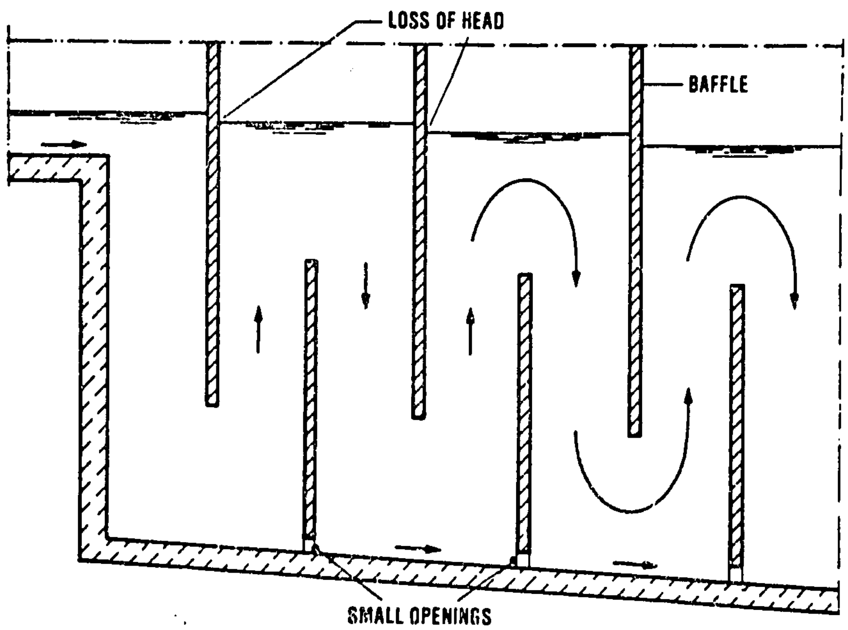
Horizontal-flow Baffled Channel Flocculator (plan)



[SOURCE: IRC, 1981b, p. 222]

FIGURE 6-3

Vertical-flow Baffled Channel Flocculator (cross-section)



[SOURCE: IRC, 1981b, p. 223]

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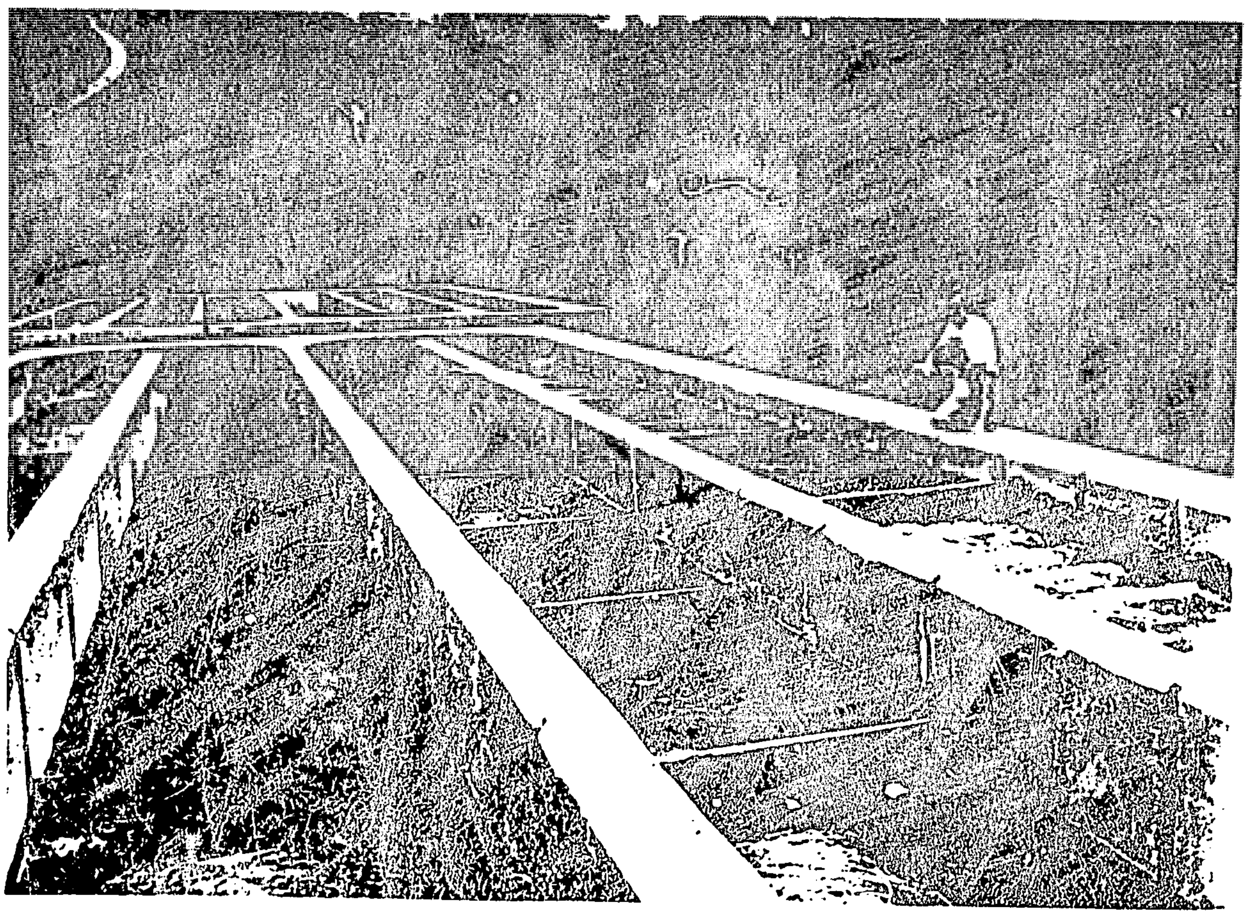
material on the chamber floors and the difficulty in removing it. To mitigate this problem, the Brazilian designs have included small openings (weep holes) in the base of the lower baffles of a size equivalent to 5% of the flow area of each chamber. The purpose is to allow the major portion of the flow of water to follow the over-and-under path created by the baffles, while a smaller portion flows through the hole, creating additional turbulence and avoiding the accumulation of material (Arboleda, 1973). An over-and-under baffled flocculator for a plant that was built in Virginia, US and has been in operation for over 40 years is shown in Figure 6-4.

The energy gradient for a horizontal flow unit is shown in Figure 6-5, revealing a relatively large head loss (h_2) across the bend (l_2) as compared to the head loss (h_1) in the channel (l_1). Recent studies (Arboleda, 1973; MacDonald and Streicher, 1977) have suggested a reliance on the velocity gradients produced in the bend for mixing, and reducing the length of the channel (l_1) so as to prevent quiescent flow. For design purposes, the head loss in the bend is approximated by the following formula:

FIGURE 6-4

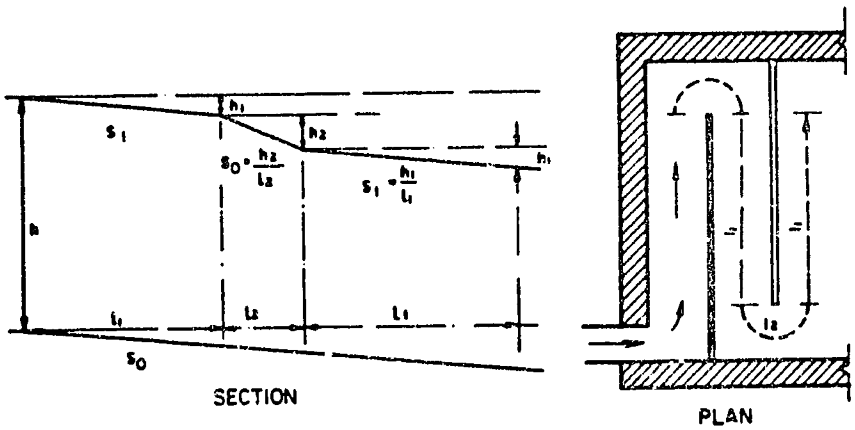
Vertical-flow Baffled Channel Flocculator for a Plant in Virginia, USA



[SOURCE: Robinson, personal communication]

FIGURE 6-5

Energy Gradient for Horizontal-flow Baffled Channel Flocculators



[SOURCE: Arboleda, 1973, p. 132]

$$H_1 = K(v^2/2g) \qquad (6\text{-}3)$$

where

H_1 = head loss (m)

v = the fluid velocity (m/s)

g = the gravitational constant (9.81 m/s^2)

K = empirical constant (varies from 2.5 to 4)

The value of K cannot be determined exactly in advance; therefore it is better to design for a low K value, because boards can always be added to the baffles if additional head loss is needed.

The number of baffles needed to achieve a desired velocity gradient for both horizontal and vertical flow units can be calculated from equations 6-4 and 6-5 below, which are adapted from formulae derived by Richter (1981).

$$n = \{[(2ut)/p(1.44+f)]\ [(HLG)/Q]^2\}^{1/3} \qquad (6\text{-}4)$$

for horizontal units

$$n = \{[(2ut)/p(1.44+f)]\ [(aLG)/Q]^2\}^{1/3} \qquad (6\text{-}5)$$

for vertical units.

where

n = number of baffles in the basin

H = depth of water in the basin (m)

L = length of the basin (m)

G = velocity gradient (sec^{-1})

Q = flowrate (m^3/sec)

t = time of flocculation (sec)

u = dynamic viscosity (kg/msec)

p = density of water (kg/m^3)

f = coefficient of friction of the baffles

a = width of the basin (m)

The water velocity in horizontal-flow and vertical-flow units generally varies from 0.3 to 0.1 m/sec. Detention time is 15 to 30 minutes (IRC, 1981b). In general, velocity gradients for both types of baffled channel flocculators should vary between 100 to 10 sec^{-1}. In addition to the foregoing design criteria, the practical guidelines enumerated in Table 6-2 should be considered in the design and construction of baffled channel flocculators, although they are somewhat general and should not be interpreted as necessarily binding in all cases.

Tapered energy flocculation in baffled channels generally is achieved by varying the spacing of the baffles, i.e. close spacing of baffles for high velocity gradients, and wider spacing for low velocity gradients. The configuration of baffles that will induce a specific tapered velocity gradient is best determined under actual plant operating conditions, by either respacing or changing the number of baffles in the flocculation basin to attain the desired head loss. Arboleda (1973) recommends a tapered velocity gradient from about 75 sec^{-1} at the inlet to 10 to 15 sec^{-1} at the outlet of the flocculators. The Cochabamba water treatment plant in Bolivia has a tapered horizontal-flow flocculator consisting of three chambers,

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TABLE 6-2: Guidelines for the Design and Construction of Baffled Channel Flocculators

A. AROUND-THE-END (HORIZONTAL FLOW)

1) Distance between baffles should not be less than 45 cm to permit cleaning.

2) Clear distance between the end of each baffle and the wall is about 1-1/2 times the distance between baffles; should not be less than 60 cm.

3) Depth of water should not be less than 1.0 m.

4) Decay-resistant timber should be used for baffles; wood construction is preferred over metal parts.

5) Avoid using asbestos-cement baffles as they corrode at the pH of alum coagulation.

* * * * *

B. OVER-AND-UNDER (VERTICAL FLOW)

1) Distance between baffles should not be less than 45 cm.

2) Depth should be 2 to 3 times the distance between baffles.

3) Clear space between the upper edge of a baffle and the water surface, or the lower edge of a baffle and the basin bottom, should be about 1-1/2 times the distance between baffles.

4) Material for baffles is the same as in around-the-end units.

5) Weep holes should be provided for drainage.

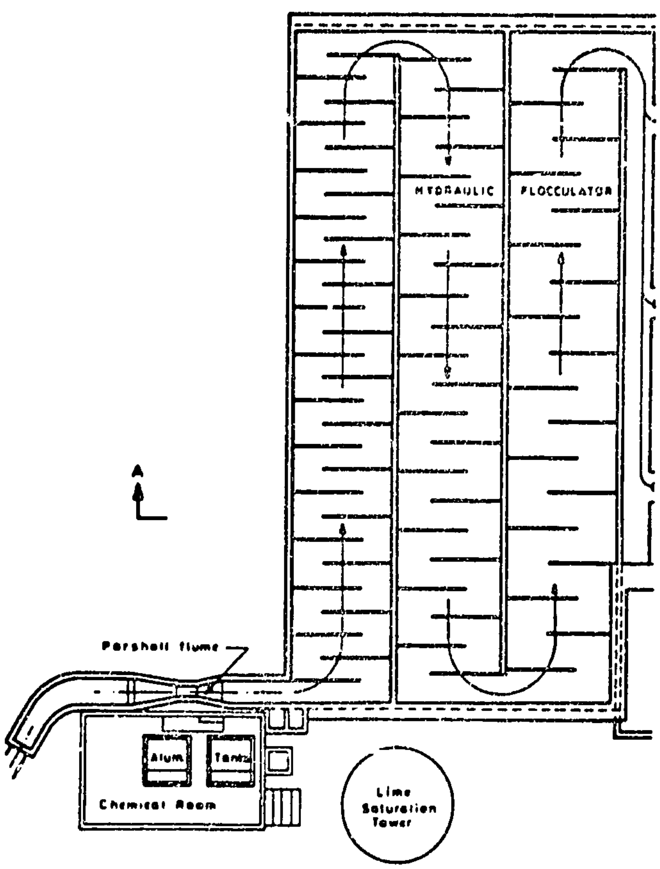
with each chamber containing baffles at different spacing, as shown in Figure 6-6.

Typical hydraulic calculations for the design of an around-the-end (horizontal-flow) flocculator are presented in Appendix B.

An innovative baffled channel flocculator with a tapered design has been installed in a plant in Oceanside, California (MacDonald and Streicher, 1977) and is illustrated in the plant layout shown in Figure 6-7. Two independent flocculation basins encompass the sedimentation and filtration units, sharing a common sidewall. Modified baffles and a sloped basin floor are arranged in such a way that the minimum water depth is at the inlet to the flocculators and the depth gradually increases to a maximum at the outlet. This results in tapered flocculation that promotes relatively high velocity gradients at the entrance, even at reduced flow rates, and decreasing gradients toward the outlet. Moreover, the overall reduced level of energy at lower flows is counterbalanced by the increased detention time. The mean velocity gradient in the two flocculator basins varies from 200 to 20 sec^{-1} at a flowrate of 38,000 m^3/day per basin, and from 208 to 8 sec^{-1} at a flowrate of 15,000 m^3/day per basin. The value of Gt over a plant flow range of 30,000 m^3/day to 60,000 m^3/day is about 65,000 to 79,000 with both flocculation channels in service. For plant flowrates less than 30,000 m^3/day, only one basin is

FIGURE 6-6

Tapered Horizontal-flow Flocculator for a Plant in Cochabamba, Bolivia



[SOURCE: adapted from Arboleda, 1976]

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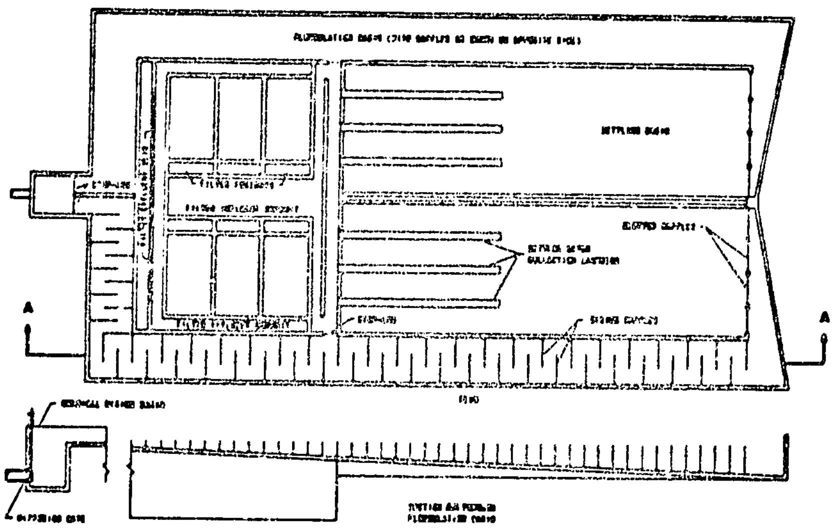
FIGURE 6-7

Tapered-energy Flocculator for the Oceanside Plant - Arcadia, California



[SOURCE: MacDonald and Streicher, 1977, p. 87]

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used. Consequently, effective flocculation can be achieved over a plant flow range of 15,000 to 80,000 m^3/day. Figure 6-8 shows the variation of the velocity gradient along the flocculator basin for high and low flowrates. Figure 6-9 shows the effect of the variation in the flow on Gt values. The flocculator performance data for the plant in Oceanside, California clearly indicate that properly designed hydraulic flocculators can operate effectively under variable flowrates; which refutes a heretofore general criticism of hydraulic flocculation systems, that maintenance of velocity gradients is not possible with changes in raw water flowrates.

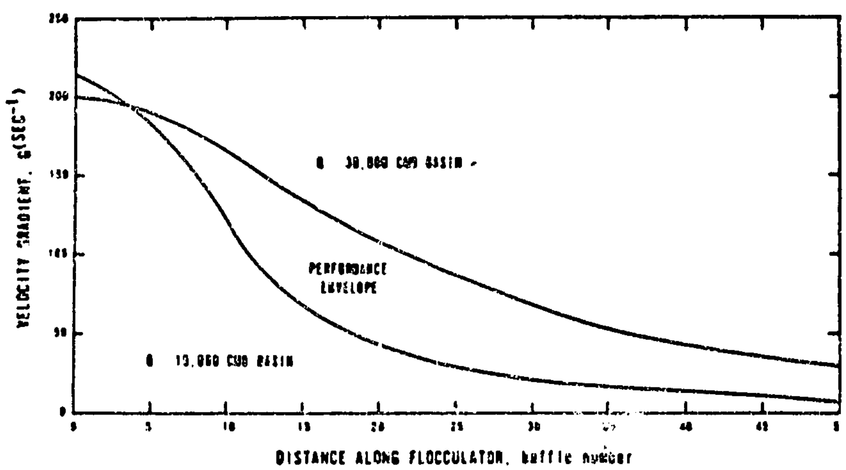
Hydraulic Jet-Action Flocculators

A less well-known type of hydraulic flocculator is one that uses the jet action of the influent water to cause agitated flow. Two types of jet-action flocculators are considered here: (1) the heliocoidal-flow type; and (2) the Alabama type. Both are presently used in small plants in Latin America, particularly in Brazil where Alabama-type flocculators are used widely (Azevedo-Netto, personal communication). The heliocoidal-flow units were extensively used in the US in the early years of this century.

Heliocoidal-flow flocculators (also called tangential-flow or spiral-flow) impart a rotational movement to the water which creates turbulence for mixing and

FIGURE 6-8

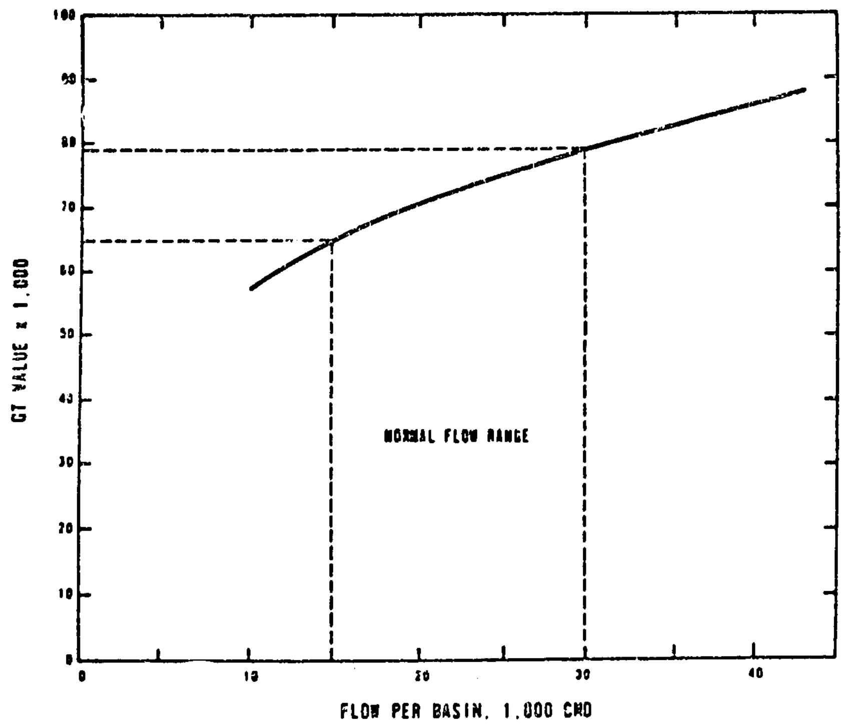
Mean Velocity Gradient Variations with Flow for the Tapered-Energy Flocculator



[SOURCE: MacDonald and Streicher, 1977, p. 88]

FIGURE 6-9

Gt Variations with Flow for the Tapered-Energy Flocculator



[SOURCE: MacDonald and Streicher, 1977, p. 88] agglomeration of the flocs. This is accomplished in a series of rectangular or cylindrical chambers by allowing a stream of water to enter tangentially into each chamber so as to cause heliocoidal flow toward the outlet. The turbulence that is created by this jet action is governed by the inlet velocities and the size and shape of the chamber. For example, a series of square chambers provide some resistance to the spiral flow of water in each of the chambers, thereby causing additional agitation. An effective design for this type of flocculator employs a series of small chambers interconnected by pipes or box conduits, carefully sized and arranged to produce the desired entry velocity for jet action. The direction of flow (upward or downward) is alternated in successive chambers. Tapered velocity gradients are easily provided for by increasing the area of the inlet opening for each successive chamber, thereby decreasing the jet action of the water and the intensity of mixing. The size of the opening for each chamber can be adjusted manually by using sluice gates, removable boards, or orifices.

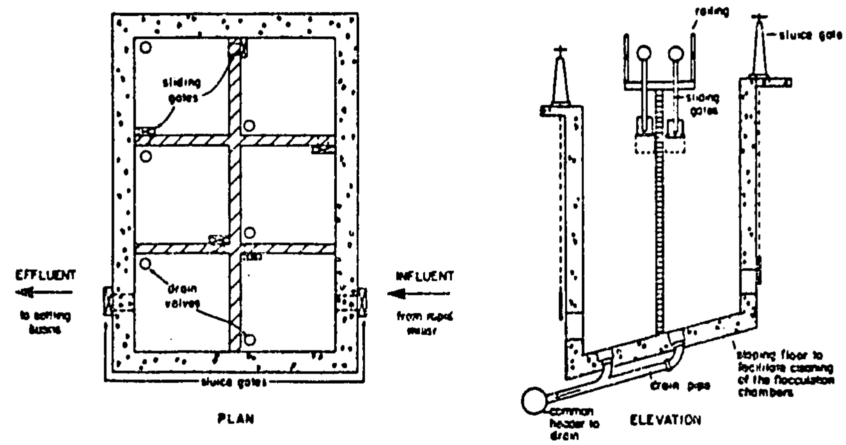
The size and number of chambers are a function of the plant flow rate and the desired time for flocculation. Five to seven basins are usually required to provide for an adequate detention time and to mitigate short-circuiting effects. For larger plants, the number may be increased by installing two or more groups of chambers operating in parallel. Recommended inlet velocities range from 0.5 m/sec to 0.7 m/sec for the first chamber to 0.1 to 0.2 m/sec for the latter chambers.

Cox (1960) designed a heliocoidal-flow flocculator for a small plant (3020 m^3/day) in Brazil with tapered-energy flocculation and flexibility in controlling the degree of agitation. The heliocoidal flocculator shown in Figure 6-10 is based on this design. Six rectangular chambers having a total volume of 60 m^3 yielded a detention time of 28 minutes for the design flow rate. Locally-made sluice gates were provided to control the size of the opening and hence the water velocity at each chamber inlet. The velocities ranged from 0.5 to 0.2 m/sec for each chamber and could be adjusted manually by turning handwheel operated sluice gates. A drain was provided for each chamber for dewatering and cleaning purposes. Arrangements were made to allow for the construction of six additional chambers to operate in parallel with the first group, so that the plant could be enlarged in the future.

An inherent shortcoming of heliocoidal-flow type flocculators, which they share with mechanical flocculators, is short-circuiting of the flow within each chamber. For mechanical flocculation, this problem has been commonly solved by installing a series of compartments within the flocculation basin. Similarly, a "staircase"-type design, developed in Brazil (see Figure 6-11) has been found

FIGURE 6-10

Heliocoidal-flow Flocculator



[SOURCE: adapted from IRC, 1980]

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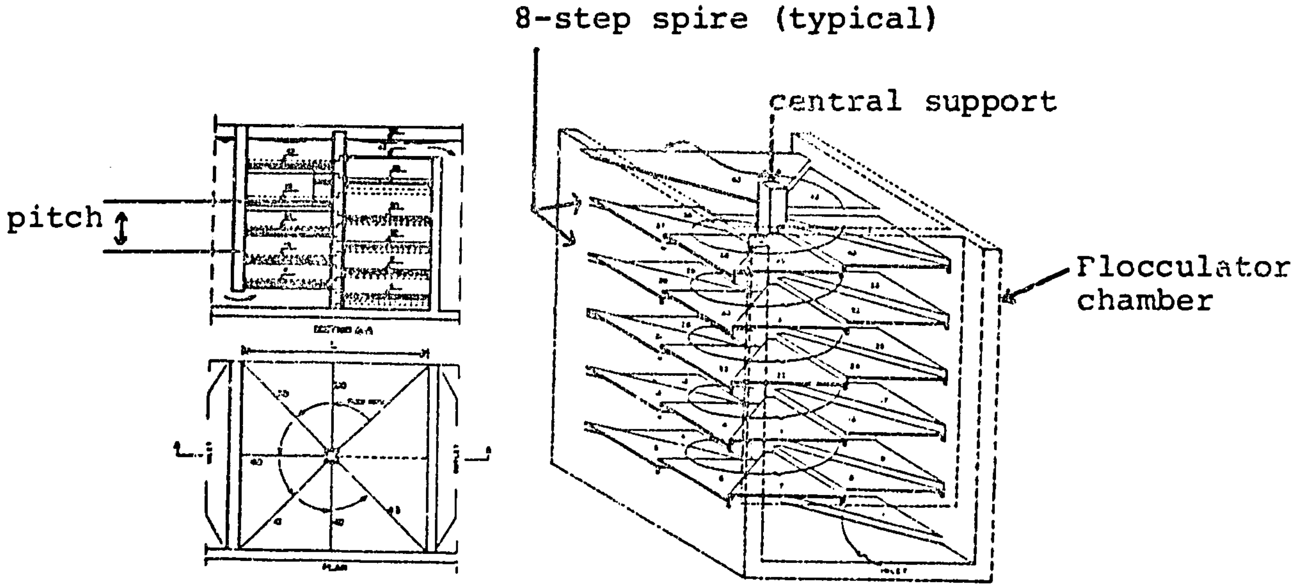
FIGURE 6-11

Staircase-type Heliocoidal Flocculator
a.) plan and section; b.) isometric view



(a) (b)

note: stairs are numbered sequentially

[SOURCE: Pinheiro, personal communication] effective in controlling short-circuiting in heliocoidal flocculation chambers as well as providing for more controlled hydraulic agitation within each chamber (Pinheiro, personal communication). This device causes a heliocoidal movement of the liquid around an axis with constant G-values at the center and periphery of any horizontal cross-sectional area of the flocculator chamber. Pinheiro developed an empirical equation for calculating G-values in staircase-type flocculators. This formula is adapted for square chambers:

$$G^2 = (2pKQ^3)/(uL^4h^3) \qquad (6\text{-}6)$$

where

G = velocity gradient (sec^{-1})

p = density (Kg/m^3)

K = friction loss coefficient (about 7.5)

g = gravitational constant (9.81 m/sec^2)

u = dynamic viscosity (kg/msec)

Q = flowrate (m^3/sec)

L = length of the side of square chamber (m)

h = pitch (m)

The staircase-type flocculator can be made from marine plywood and is assembled like a spiral staircase with the treads around a central column. The flight of 4 or 8 steps corresponds to a spire, and all treads are equal trapeziums; therefore each rise is 1/4 or 1/8 of the pitch (see Figure 6-11b). The inlet and the outlet of the chamber are positioned opposite each other. With some modifications, staircase-type flocculators may be retrofitted in conventional heliocoidal-flow flocculation chambers. Hydraulic calculations for staircase flocculators are presented in Appendix B.

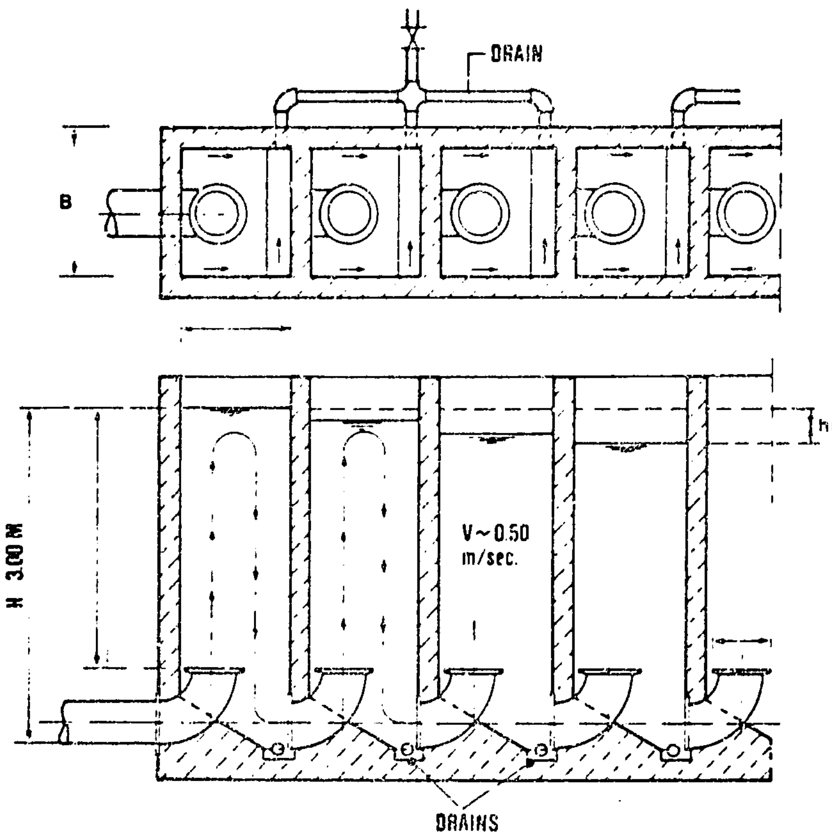
The Alabama-type flocculator is illustrated in Figure 6-12. The jet action is provided in each chamber via a cast iron pipe with its outlet turned upwards. For effective flocculation, the outlet should be placed at a depth of about 2.5 meters below the water level. Common design criteria are listed below:

Rated capacity per unit chamber	25-50 l/sec per m^2
Velocity at turns	0.40-0.60 m/sec
Length of unit chamber (L)	0.75-1.50 m
Width (B)	0.50-1.25 m
Depth (h)	1.50 to 2.50 m
Detention time (t)	15 to 25 minutes

Table 6-3 provides practical guidance for the design of Alabama-type flocculators. The head loss with this type of flocculator is estimated at two velocity heads per chamber; generally about 0.35 to 0.50 m of head loss for the entire unit. Velocity gradients range from 40-50 sec^{-1}. Arrangements should be made for draining each chamber, as accumulated material tends to collect at the bottom and must be removed occasionally.

FIGURE 6-12

"Alabama"-type Flocculator



[SOURCE: IRC, 1981b, p. 224]

TABLE 6-3: Guidance for "Alabama"-type Flocculator Design

Flow Rate Q (m^3/day)	Width B (m)	Length L (m)	Diameter D (mm)	Unit Chamber area (m^2)	Unit Chamber volume (m^3)
864	0.60	0.60	150	0.35	1.1
1730	0.60	0.75	250	0.45	1.3
2590	0.70	0.85	300	0.6	1.8
3460	0.80	1.00	350	0.8	2.4
4320	0.90	1.10	350	1.0	3.0
5180	1.00	1.20	400	1.2	3.6
6050	1.05	1.35	450	1.4	4.2
6910	1.15	1.40	450	1.6	4.8
7780	1.20	1.50	500	1.8	5.4
8640	1.25	1.60	500	2.0	6.0

[SOURCE: adapted from IRC, 1981b, p. 225]

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Gravel-Bed Flocculators

The gravel-bed flocculator provides a simple and inexpensive design for flocculation in small water treatment plants (less than 5000 m^3/day capacity), and has been tested experimentally and employed successfully in several plants in India (Kardile, 1981). The packed bed of gravel provides ideal conditions for the formation of compact settleable flocs due to continuous recontacts provided by the sinuous flow of water through the interstices formed by the gravel. The velocity gradients that are introduced into the bed are a function of (1) the size of the gravel; (2) rate of flow; (3) cross-sectional area of the bed; and (4) the head loss across the bed. The direction of flow can be either upward or downward, and is usually determined from the design and hydraulic requirements of other process units in the plant. A unique characteristic of this type of hydraulic flocculator is its ability to store agglomerative flocs within the interstices or to settle flocs on top of or below the gravel bed (depending on the direction of flow) due to the sudden drop in velocity as the flow of water emerges from the bed. Moreover, the sludge storage capabilities of gravel-bed flocculators make them ideal pretreatment units prior to filtration in small plants, often eliminating the need for a separate sedimentation step (see Chapter 8, "Upflow-Downflow Filtration").

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Velocity gradients and head losses in gravel-bed flocculators can be estimated from the following formulae (adapted from SANEPAR, 1979):

$$G = [(h_1 pgQ)/(ufV)]^{1/2} = [(h_1 pg)/uft)]^{1/2} \tag{6-7}$$

$$h_1 = av + bv^2 \tag{6-8}$$

$$a = [0.162(1-f)^2 u]/(\phi^2 D^2 f^3 p) \tag{6-9}$$

$$b = [0.018(1-f)]/(\phi D f^3) \tag{6-10}$$

where

G = velocity gradient (sec^{-1})

h_1 = head loss (cm)

a, b = coefficients used in Eq. 6-8

g = gravitational constant (980 cm/sec^2)

Q = flow rate (cm^3/sec)

u = dynamic viscosity (kg/msec)

p = specific gravity of water (kg/m^3)

f = porosity ($\cong$0.4)

V = volume of gravel bed (cm^3)

v = face velocity (cm/sec)

ϕ = shape factor ($\cong$0.8)

D = average size of gravel (cm)

It should be noted that Equation 6-7 is similar to equations 5-1 and 6-1. A design for a gravel-bed flocculator using the above equations is presented in Appendix B.

When greater accuracy is desired, G-values may be determined from bench scale experiments. Plastic cylinders are filled with the desired gravel medium at the same depth

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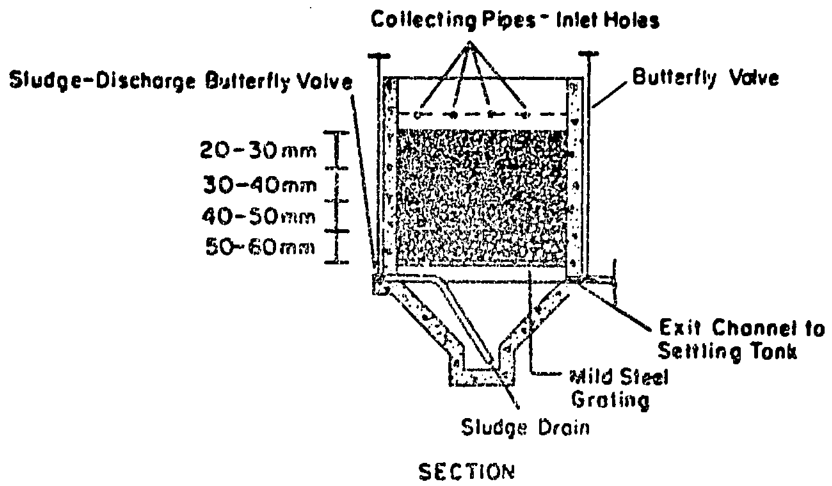
as the full-scale gravel-bed flocculator, and arrangements are made for measuring head loss at several points along the length of the cylinder. After sufficient head loss data are collected for a range of flows, the corresponding velocity gradients can be calculated from equation 6-7.

Tapered velocity gradients are achieved in gravel-bed flocculators by changing the cross-sectional area of the bed and/or by grading the bed with different sized layers of gravel. The downward flow unit in Figure 6-13 is comprised of a graded gravel bed ranging in size from 20 mm to 60 mm from top to bottom inside a concrete masonry chamber, and supported on mild steel grating. The hopper bottom in the chamber has 45° slopes, and is used to drain sludge under hydrostatic pressure.

The upward flow unit, shown schematically in Figure 6-14, combines two sizes of layered gravel (5 to 10 mm and 10 to 20 mm) with sections of increasing cross-sectional area to produce the desired tapering. The velocity gradients range from 846 sec^{-1} at the inlet (where rapid mixing occurs), to 31 sec^{-1} in the uppermost and largest section for a flow rate of 270 m^3/day (see Appendix B for corresponding hydraulic calculations). The terraced shape of the flocculator is formed out of mild steel, and is protected by corrosion proof paint and supported by horizontal rods attached to an outer concrete chamber. This design has been used in package plants in India (see Chapter 9).

FIGURE 6-13

Downward-flow Gravel Bed Flocculator



[SOURCE: adapted from Kardile, 1981, p. 226]

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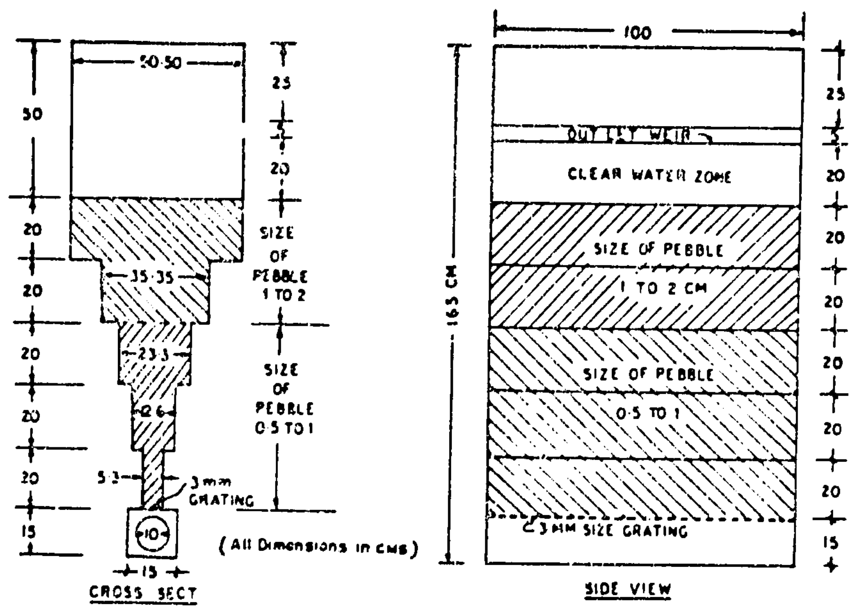
FIGURE 6-14

Upward-flow Gravel Bed Flocculator



[SOURCE: Bhole, 1981, p. 317]

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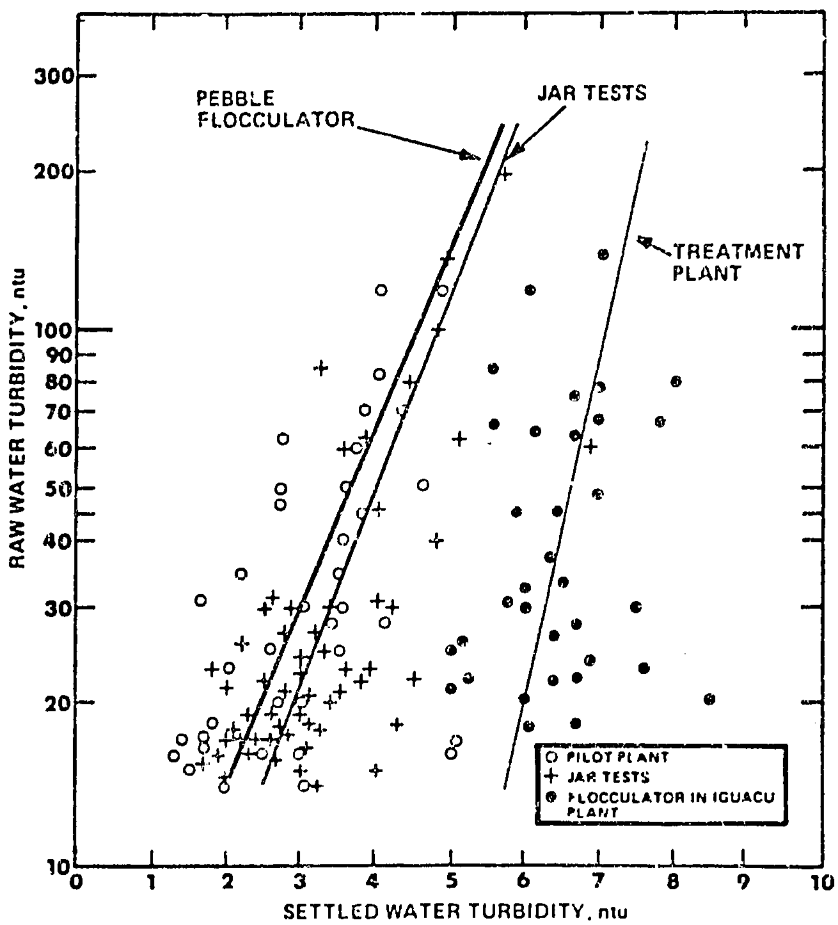
Flocculation time can be reduced considerably by using gravel beds because the entire bed is effective in the formation of sizeable flocs and there is very little short-circuiting. Three to five minutes flocculation in the gravel bed is equivalent to 15 minutes in jars under laboratory conditions, and to 25 minutes in noncompartmented plant flocculation basins, as revealed in the graph of Figure 6-15 (Wagner, 1982; Richter, 1981). Depth of the gravel bed generally varies from 1.5 to 3 m. Flocculated water may be conveyed from the flocculation chamber to the settling tanks via submerged perforated pipes or channels. The sedimentation step is often deleted in small plants, and the flocculated water is applied directly to the filter.

The main problem with gravel-bed flocculators is likely to be one of fouling, either by intercepted flocs or biological growth in the gravel. Therefore, sludge collection and removal is an important consideration in the design of such units. For downward flow units hopper bottoms, such as that depicted in Figure 6-13, drain the sludge by hydrostatic pressure. Upward flow units often rely on a perforated drainage pipe grid located just above the top of the bed for removing the sludge that is deposited on the surface of the gravel. Both types of flocculator units should include arrangements for draining the water from the flocculator chamber to waste, and backwashing

FIGURE 6-15

Comparison of Results of Gravel Bed (pebble) Flocculation in the Pilot Plant with Results of Jar Tests with the Full-scale Plant Flocculator at the Iguacu Plant - Curitiba, Brazil



[SOURCE: adapted from Richter, 1981]

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capabilities to completely remove sludge settled within the bed.

Gravel bed flocculators have proven to be simple, low-cost and an effective method of flocculation for several small water treatment plants in India (Kardile, 1981) and have been used recently in modular plants in Latin America (CEPIS, 1982). They have also been installed in low-cost package water treatment plants designed and manufactured in India (Bhole, 1981). Plant designs that employ gravel-bed flocculators are described in Chapter 8 ("Upflow-Downflow Filtration"), and Chapter 9 ("Package Water Treatment Plants").

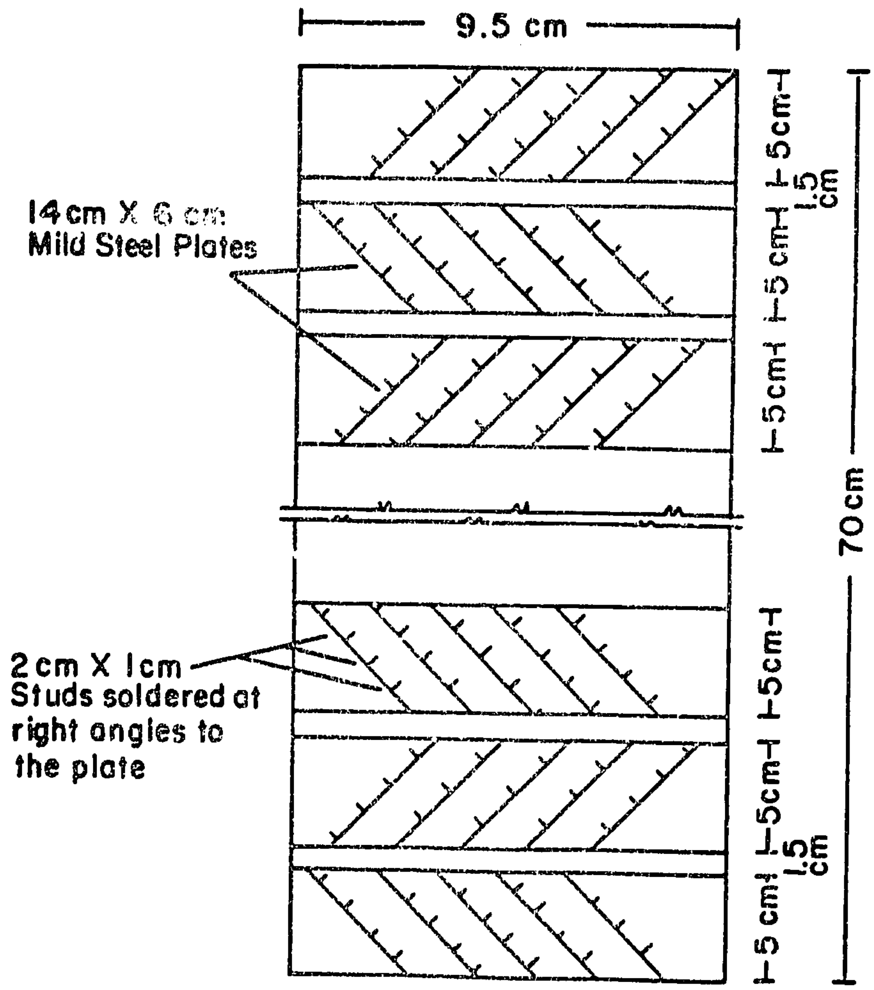
Surface-Contact Flocculators

Surface-contact flocculators have been studied experimentally in India (Bhole and Ughade, 1981) as a means to overcome the inherent problem of sludge choking in gravel-bed flocculators, which increases the head loss over time in such systems so that periodic cleaning of the gravel bed is necessary.

Surface-contact flocculators consist mainly of studded plates placed in a zig-zag form along the direction of flow, as shown in Figure 6-16. The experimental flocculator used in the Indian study was comprised of 55 mild steel plates, 14 cm x 6 cm in size, arranged in eleven rows of five plates each. These plates were fixed at 45° to a base plate in a

FIGURE 6-16

Surface-contact Flocculator - India



[SOURCE: Bhole and Ughade, 1981, p. 180] zig-zag fashion. Each plate was studded with 14 strips, each 2 cm x 1 cm in size.

The flocculator was tested in a continuous down-flow system, with flow rates ranging from 5 $m^3/m^2/hr$ to 25 $m^3/m^2/hr$ and turbidities ranging from 50 to 1600 NTU. The results showed surface-contact flocculators to be most effective for low turbidity waters and low rates of flow. The build-up of head loss was negligible, indicating that sludge choking was not a problem. The authors concluded sludge choking was not a problem. The authors concluded surface waters containing about 100 NTU turbidity, with flow rates as high as 25 $m^3/m^2/hr$. Presently, no information is available on the effectiveness of these units under plant operating conditions.

VII. SEDIMENTATION

The sedimentation process in water treatment is responsible for the settling and removal of suspended material from water. Most commonly, it is used in conventional treatment for sedimentation of flocculated particles prior to filtration. The removal efficiency in the sedimentation basin determines the subsequent loadings on the filters and, accordingly, has a marked influence on their capacity, the length of filter runs, and the quality of the filtered water. The two major classifications for the design of sedimentation basins are (1) horizontal-flow units, and (2) upflow units. The design of both types of units involves such factors as shape, number of basins, dimensions, velocity, and direction of flow, detention time, volume of sludge storage, method of sludge removal, inlet and outlet arrangements, and the characteristics of the incoming flocculated water.

The horizontal-flow sedimentation basin has performed admirably in numerous water treatment plants in the United States and other parts of the world for decades and is still advocated by water treatment experts because of its efficiency and inherent simplicity (Sanks, 1978; Hudson, 1981; Smethurst, 1979). The use of such units has diminished somewhat in the United States, though, due to the development of proprietary-upflow clarifiers, such as solids-contact reactors and slurry-recirculation units that combine the processes of mixing, flocculation, and sedimentation into a single unit. The advantages of such units are largely economic, i.e. by combining the pretreatment processes that precede filtration, substantial savings can be realized in construction costs and manpower requirements. Upflow clarifiers perform quite well under suitable conditions and skilled supervision, so long as their hydraulic capacity is not exceeded. When upflow clarifiers are overloaded, sludge escapes from the blanket in large volumes and clogs the filters, interfering with the entire treatment process. For developing countries, horizontal-flow tanks without mechanical sludge removal are much to be preferred, because they require no importation of equipment, and labor for cleaning the tanks is readily available. Equally important, horizontal-flow tanks can be overloaded with little deleterious effects on subsequent filtration, as most of the settleable solids will still settle out. Overloading of plants is a chronic condition in developing countries.

The principles governing the design and construction of horizontal-flow sedimentation basins are well documented in standard texts (AWWA, 1971; Cox, 1965; Fair, Geyer, & Okun, 1968; Hudson, 1981; IRC, 1981b; Sanks, 1978; Smethurst, 1979). The topics covered on horizontal-flow sedimentation include design criteria, inlet and outlet arrangements, methods for sludge removal, and the application of tube and inclined plate settling. In addition, upflow-type clarifiers are presented briefly, as such designs may be appropriate in places where large horizontal-flow tanks are impractical.

Horizontal-Flow Sedimentation

Horizontal-flow sedimentation is a gravity separation process where a settling basin provides a quiescent environment that enables particles having specific weights greater than water to settle to the bottom of the tank. A well-designed horizontal-flow sedimentation basin can remove up to 95% of raw water turbidity following effective coagulation and flocculation; the remaining turbidity is removed in the filters. Rectangular horizontal-flow clarifiers without mechanical sludge removal are advantageous for communities in developing countries because of their simplicity, and ability to adapt to various raw water conditions, such as sudden changes in turbidity or excessive flow rates. Circular-shaped basins are not recommended, since their only advantage over rectangular basins is more efficient mechanical sludge removal (utilizing central-drive scrapers), at the expense of less efficient settling. Manual sludge removal is preferred in developing countries over the importation of mechanical sludge removal equipment, as the latter is difficult to maintain under the technical and climatic conditions prevalent in those countries, and more costly than employing laborers to clean the tanks manually. Manual cleaning is most readily accomplished with rectangular basins. Many plants in the US still use manually cleaned horizontal-flow rectangular basins.

A rectangular horizontal-flow sedimentation basin is shown in Figure 7-1. Flocculated water is distributed uniformly across the inlet zone through diffusers, such as perforated inlet baffles. The water slowly traverses the length of the basin, depositing settled floc on the tank bottom, forming a sludge layer in a fashion outlined by the sludge profile in Figure 7-1. The clarified supernatant is collected by outlet weirs or submerged launders. The sloping floor of the bottom facilitates manual cleaning and drainage of sludge, usually by means of high pressure hoses or fixed nozzles on the basin floor.

There are several advantages of horizontal-flow units over upflow units:

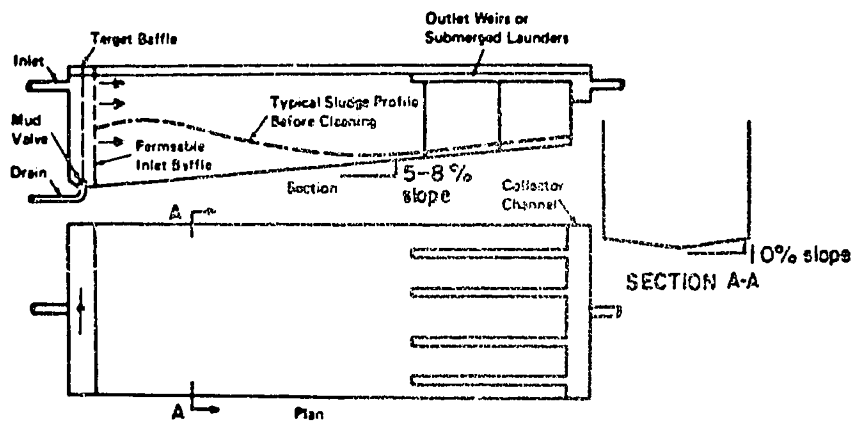
1) the process is more tolerant of hydraulic and quality variations;

2) the process gives predictable performance under most operational and climatic conditions;

3) the process "scales-up" very well, and is most economic for larger plants;

FIGURE 7-1

Conventional Horizontal-flow Settling Basin



[SOURCE: adapted from Hudson, 1981, p. 136]

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4) the process works exceptionally well when silt loads are very high;

5) construction costs are low, permitting oversizing; and

6) operation and maintenance is simple.

Although horizontal-flow units may be more expensive to construct than upflow clarifiers in the industrialized countries (because of lower surface loadings which require larger-sized tanks), this is not the case in developing countries where these units can be built quite cheaply using local materials, such as concrete or masonry, and lower-cost labor. Upflow equipment would need to be imported, in general, from foreign manufacturers.

Design Criteria

The design of sedimentation basins is governed by three basic criteria: 1) the quantity of water to be treated; 2) the selected detention period; and 3) the selected surface loading rate (or overflow rate). The surface loading rate is defined as the ratio between the influent flow rate and the surface area of the tank and can be expressed in units of flow rate per unit of basin surface area (e.g. $m^3/ hr/m^2$). This is equivalent to a velocity; hence some design books prefer to use settling velocity as a loading parameter (Smethurst, 1979; Hudson, 1981). The basic formulae that pertain to sedimentation basin design are:

$$t = 24\ V/Q \qquad (7\text{-}1)$$

$$S_o = H/T \qquad (7\text{-}2)$$

$$S_o = Q/BL \qquad (7\text{-}3)$$

where

S_o = surface loading rate or settling velocity ($m^3/m^2/hr = m/hr$)

t = detention time (hr)

Q = flow rate (m_3/day)

V = basin volume (m^3)

These formulae can be used in conjunction with certain graphical methods (e.g., cumulative frequency distributions) to determine settling velocities for the settleable particles in the raw water. Settling data may be obtained by running preliminary bench scale experiments utilizing plastic cylinders equal in depth to the proposed basin, with draw-off points at different levels, and filled with test samples of the raw water. Samples are taken at regular time intervals to measure the turbidity at various depths in the cylinder, which is an indication of the rate of settling. If the period of settling is short, and a distinct separation forms between the upper clarified zone and the lower zone of settled solids, then flocculation is probably not necessary. If, however, the period is relatively long, and the two zones are not well-defined, then it is likely that colloidal material is present, and flocculation is essential. The settling test should be repeated, using coagulated water, after jar tests have been run to determine the optimum dose of coagulants and, if necessary, coagulant aids. Settling test procedures are outlined in some detail by the IRC (1981b).

An inherent assumption in the settling test is that the settling process is not hindered by density currents, eddies, temperature changes, or other conditions found in actual practice. Practical experience has shown that a discrepancy exists between the design values predicted by theoretical formulae, and bench scale testing, and the design values found most effective in practice. For example, the mean detention time is the time required for a particle of water to flow through the basin and is computed by dividing the volume of the basin by the rate of flow through it, the theoretical detention period (Eq. 7-1). The mean and theoretical detention periods are identical. Short-circuiting, however, the extent of which is affected by density currents and eddies, inlet and outlet structures, baffles, and the shape and dimensions of the basin, makes the observed "flowing-through period" for some of the particles of water in the basin shorter (and some longer) than the theoretical detention time. Because removals are a decreasing function of time, the removals will be considerably less with short-circuiting than would occur if all the water particles were held for the mean detention period. Although most designs try to minimize short-circuiting in sedimentation basins, it cannot be eliminated completely. Hence, the design of sedimentation basins rests largely on experience and should integrate the results from experimental settling tests with established guidelines which have proven successful in practice.

One such rule of thumb guideline has been suggested by Smethurst (1979). In order to account for the short-circuiting phenomena in horizontal-flow basins when conducting bench scale settling tests, the time it takes for the average suspended solids of the water at all draw-off points above the sludge zone to fall to a concentration of 2 mg/l should be multiplied by a factor of safety of 3 to arrive at the nominal detention time of the proposed settling basin.

Table 7-1 lists recommended surface loading rates (settling velocities) and detention times for the different conditions likely to be encountered in practice. The stated design values vary considerably (by a factor of two over their entire range) depending on the type of unit under consideration. Table 7-1 also reveals that effective mixing and flocculation prior to sedimentation (condition D) can substantially reduce required detention times and increase surface loadings, thereby enabling the design of smaller, less costly settling basins. Design parameters for several water treatment plants in Latin America are tabulated in Tables 7-2 and 7-3, and include specifically plant capacity,

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TABLE 7-1: Design Guidelines for Horizontal-flow Settling Basins

TYPE	DESCRIPTION	SURFACE LOADING RATE (SETTLING VELOCITY) (m/day)	DETENTION PERIOD (hours)
A	small installations with precarious operation	20 - 30	3 - 4
B	installations planned with new technologies[a] and reasonable operation	30 - 40	2-1/2 - 3-1/2
C	installations planned with new technologies[a] and good operation	35 - 45	2 - 3
D	large installations with new technologies[a] and excellent operation, with provisions for adding coagulant aids whenever necessary	40 - 60	1-1/2 - 2-1/2

[a]Properly designed hydraulic rapid-mix and flocculation units.

[SOURCE: adapted from Azevedo-Netto, 1977, p. 780]

TABLE 7-2: Design Parameters for Horizontal-flow Settling Basins in Brazil

LOCATION OF WATER TREATMENT PLANT	CAPACITY (m^3/sec)	SETTLING VELOCITY (m/day)	DETENTION PERIOD (hours)
Guarau, Sao Paulo	33.0	40.5	2.97
Rio das Velhas, Belo Horizonte	9.0	48.9	2.13
Rio Descoberto, Brasilia	6.0	41.1	2.18
Campinas	2.1	34.1	3.0
T. Ramos Sao Paulo[a]	2.0	58.8	2.0

[a]Experimental operation

[SOURCE: Azevedo-Netto, 1977, p. 781]

TABLE 7-3: Efficiency of Horizontal-flow Settling Basins in Colombia (1959)

LOCATION OF WATER TREATMENT PLANT	DETENTION PERIOD (hours)		SETTLING VELOCITY (m/day)		TEMPERATURE (°C)		INFLUENT TURBIDITY (NTU)		EFFLUENT TURBIDITY (NTU)	
	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.
Agua de Dios	7.05	5.5	13.1	10.2	22	25	50	2	10	1
Cali	5.05	3.92	22.2	17.3	18	2	600	5	8	2
Pasto	3.96	3.1	27.2	21.3	14	14	120	5	4	0.9
Pereira (new)	2.66	2.13	35.4	28.2	18.5	18.5	130	7	9	4
Pereira (old)	4.1	2.8	27.6	18.8	17.7	17.8	480	5	9	4
Ipiales	10.65	3.36	18.4	5.8	11	11	200	4	30	1
Santa Marta	3.31	1.83	50.2	27.6	28	28	4690	59	6	2
Zipaquira	13.9	5.9	12.4	5.3	13.4	13	4	1	2.0	0.2

[SOURCE: Arboleda, 1973, p. 214]

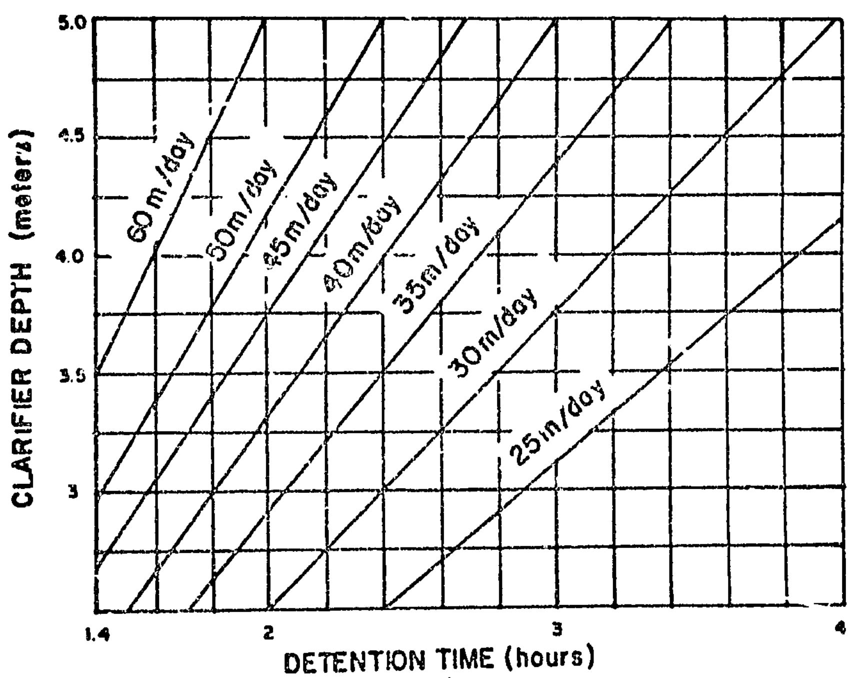
surface loadings, and detention times for settling basins. In addition, Table 7-3 lists plant operating temperature ranges and settling efficiency, the latter obtainable by comparing raw water versus settled water turbidity.

The remaining design criteria are concerned primarily with mitigating the problems of turbulence, short-circuiting and bottom scour (i.e. disturbance of the sludge layer). A basin depth of 3 meters is recommended to allow for sludge deposits and storage in the basin. The relationship among basin depth, detention time, and surface loadings are revealed in the graph of Figure 7-2. For example, a detention time of 3 hours and an overflow rate of 30 m/day would fix the clarifier depth at 3.75 meters. To reduce the likelihood of short-circuiting, a length to width ratio (L/B) of 3 or more is recommended. Horizontal flow velocities are fixed by these constraints, and should range from 4 to 36 m/hour. It should be noted that these velocities are not uniform across the basin cross-section, due to the influence of drag from the floor and walls of the basin. Basin drag is also a contributing factor in the formation of density currents (Arboleda, 1973; Hudson, 1981; Sanks, 1979). The number of basins that should be selected for a particular plant is influenced by (1) the effect upon the production of water if one basin is removed from service; and (2) the largest size which can be expected to produce satisfactory results. The AWWA publication Water Quality and Treatment

V 3 7

FIGURE 7-2

Detention Times for Different Clarifier Depths and Overflow Rates



[SOURCE: Arboleda, 1973, p. 219]

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(1971) recommends a minimum of two basins, and more where feasible, to mitigate the effects of velocity and detention period in the remaining basins, when one basin is removed from service for cleaning.

Inlet Arrangement

Inlet arrangements should be so designed that the flocculated water entering the basin is distributed uniformly across the full cross-sectional area of the inlet zone without causing excessive turbulence which would break up the floc. It is important to take into account manifold hydraulics to attain good distribution of flows among parallel basins. The proper sizing of ports and manifolds is much to be preferred over regulating devices. Practical hydraulics for both dividing-flow and combining flow manifolds is covered by Hudson (1981).

An efficient type of inlet arrangement employs perforated baffles, in a diffusion wall following target baffles, as shown in Figure 7-3. Hudson suggests velocities through the perforated baffles of about 20 to 30 cm/sec. The head loss through the ports is estimated to be 1.7 times the velocity head ($V^2/2g$). The perforated baffle wall is usually constructed with concrete, but timber baffles or brick masonry may also be used. Four basic requirements should be met in the design of perforated baffles for settling basins (Hudson, 1981):

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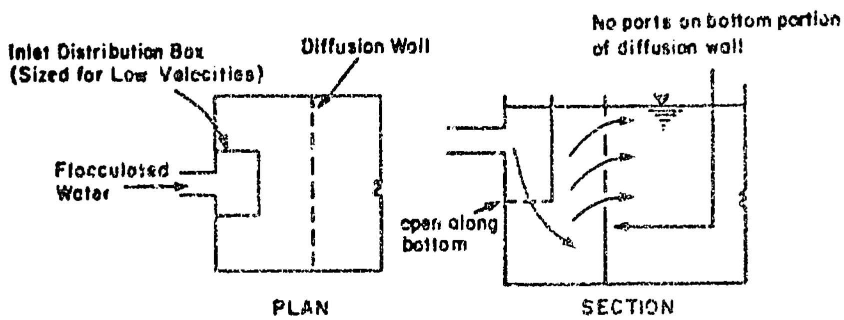
FIGURE 7-3

Inlet Arrangement Consisting of a Flow-distribution Box,
Followed by a Diffusion Wall



[SOURCE: Hudson, personal communication]

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1) The velocity through the ports should be about four times higher than any approaching velocities in order to equalize flow distribution both horizontally and vertically.

2) To avoid breaking up floc, the velocity gradient through inlet conduits and ports should be held down to a value close to or a little higher than that in the last portion of the flocculator.

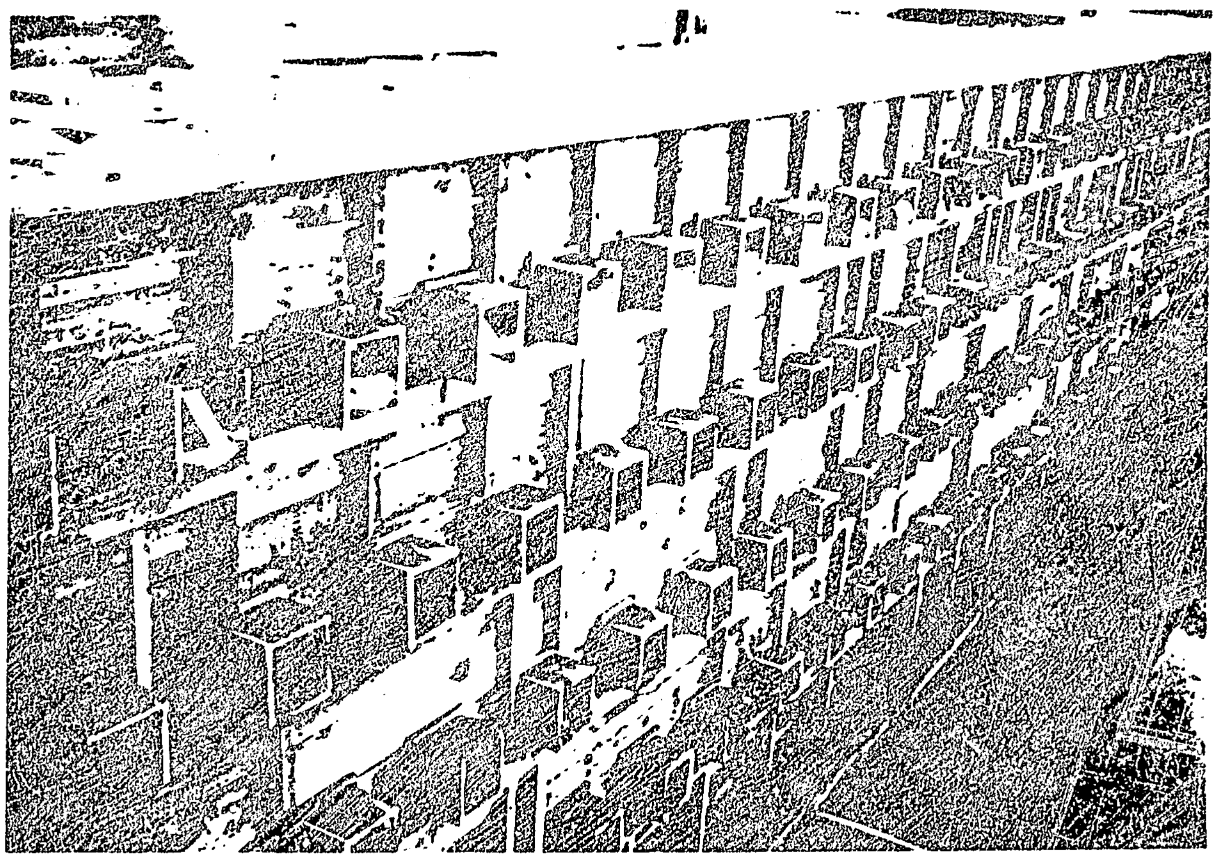
3) The maximum feasible number of ports should be provided in order to minimize the length of the turbulent entry zone produced by the diffusion of the submerged jets from the ports in the perforated baffle inlet.

4) The port configuration should be such as to assure that the discharge jets will direct the flow towards the basin outlet.

To ensure proper dimensioning of the ports for timber baffles, tubular inserts made of plastic or of wood construction can be fastened to the openings of the timber wall. The latter type of insert is shown in Figure 7-4 which is a photograph of a timber diffusion wall for the Guandu plant in Rio de Janeiro, Brazil. For masonry walls, a checkered configuration may be constructed by intentionally leaving bricks out of the wall at certain spacings (Figure 7-5). After plant start-up, loose half bricks can be added to improve the distribution of the incoming water, if necessary.

FIGURE 7-4

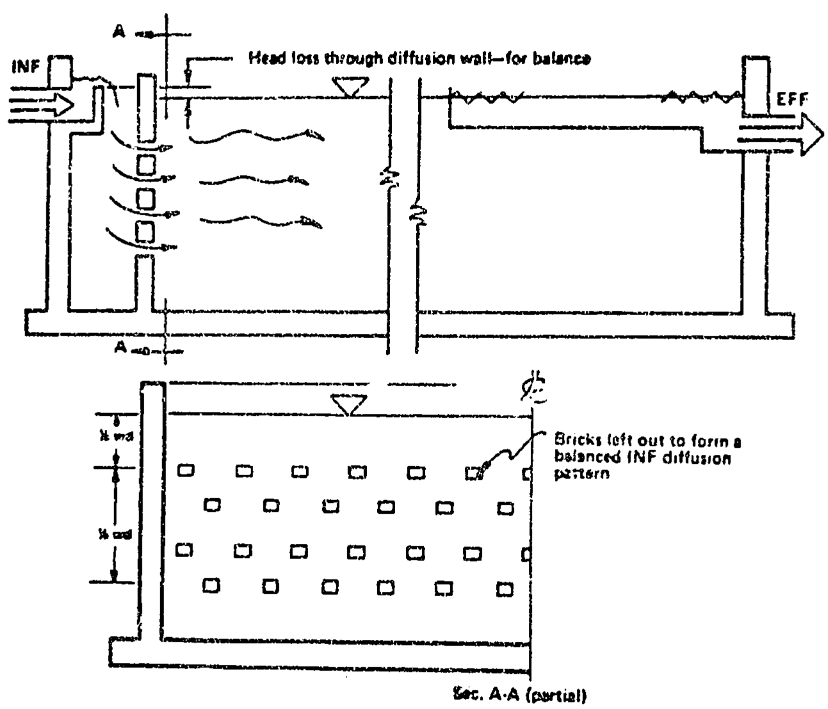
Timber Diffusion Wall at the Guandu Plant -
Rio de Janeiro, Brazil



[SOURCE: Hudson, personal communication]

FIGURE 7-5

Checkerwork Influent Diffusion Wall



[SOURCE: Sanks, 1979, p. 154]

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Outlet Arrangement

Weirs or perforated launders are the most common structures for withdrawing the effluent water from the basin. In order to prevent high velocities of approach and disturbance of the sludge layer, great weir lengths should be employed, extending up to half or more of the length of the basin, if necessary. The following formula is useful in determining an acceptable weir length:

$$L = 4.8Q/HS_o$$

where

L = combined weir length (m)

Q = flow rate (m^3/day)

H = depth of tank (m)

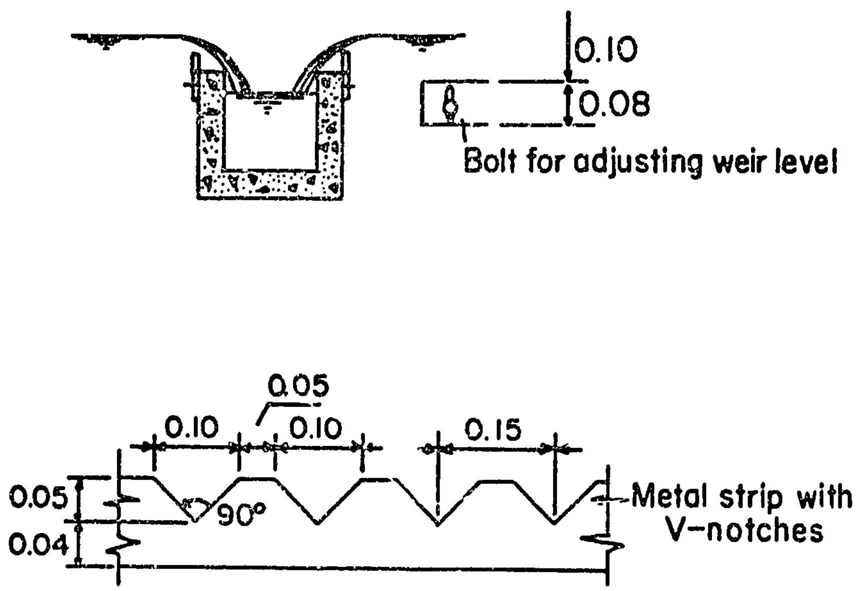
S_o = settling velocity (m/sec)

The outlet weirs or launders may be arranged either parallel to or transverse to the direction of flow in the basin. A center-to-center distance of one to two times the depth of the tank is reasonable for outlet conduit channel spacing.

Adjustable V-notched weirs are convenient for ensuring uniform flow throughout the collecting trough, especially when low overflow rates are used. They are constructed from metal strips containing V-notches about 5.0 cm deep and 15 to 30 cm apart; which are fastened with bolts to the concrete wall of the collecting trough (Figure 7-6). However, overflow weirs must be leveled accurately, which

FIGURE 7-6

Adjustable V-notch Weirs Attached to Effluent Launders
(dimensions in meters)



[SOURCE: Azevedo-Netto, 1977]

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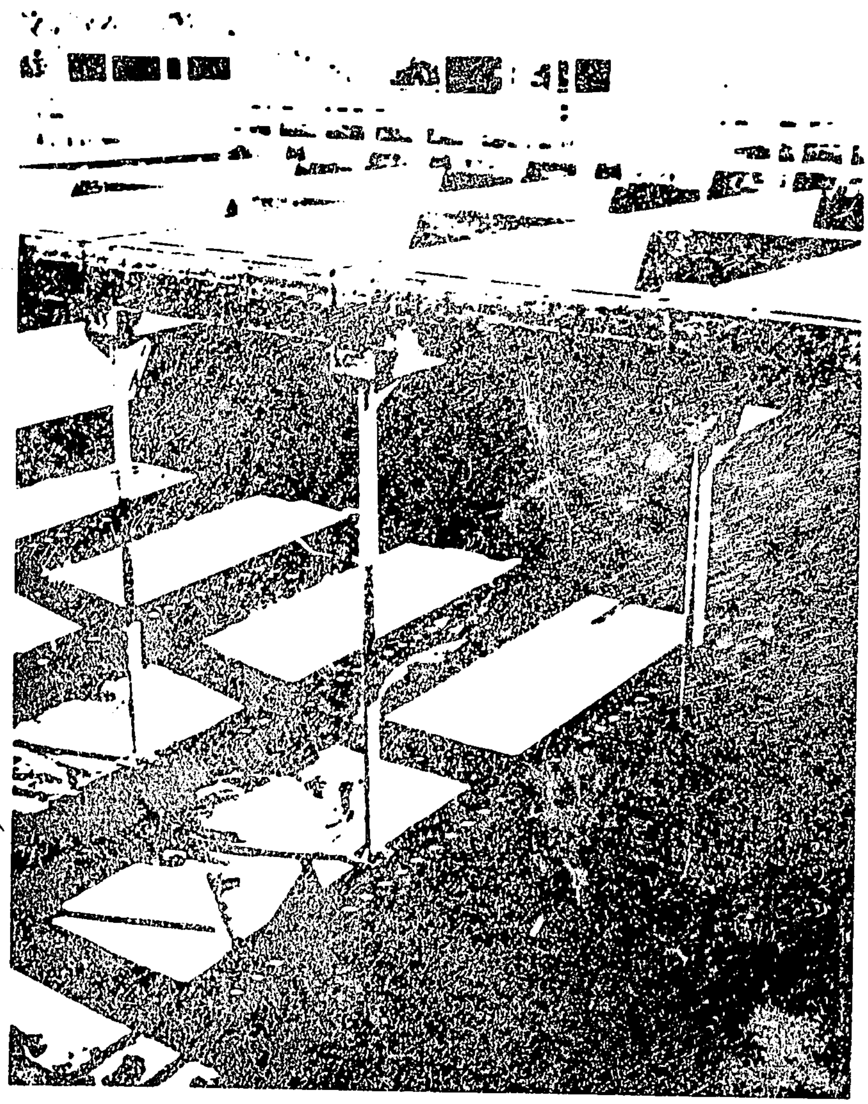
may be difficult in places where skilled plant personnel are not available. Perforated launders, on the other hand, have ports submerged 30 to 90 cm below the surface, and hence do not require precise leveling Submerged launders are useful in preventing floating debris from entering the outlet conduits and can readily handle small changes in water levels in the basin. Storage in the settling basin is often used to permit some temporary differences between inflow to the plant and the discharge from the plant. This cannot be done when overflow weirs are used in the basin outlet. Perforated launders may be tapered to prevent velocities from increasing too much along them. A perforated launder for a large settling basin in the Guandu Plant in Rio de Janeiro, Brazil is shown in Figure 7-7.

Manual Sludge Removal

The sludge collection and removal mechanisms that are commonly employed in horizontal-flow sedimentation basins in industrialized countries, such as chain and sprocket scrapers or vacuum-type systems, are not practical in most developing countries. Manual, rather than mechanical, sludge removal is preferred because it does not require imported equipment nor spare parts, and the labor required is low-cost and abundant. Although manual sludge removal requires the periodic shut-down of a basin while it is being cleaned, this should not pose a problem when two or more basins are available.

FIGURE 7-7

Perforated Effluent Launders for the Guandu Plant - Rio de Janeiro, Brazil

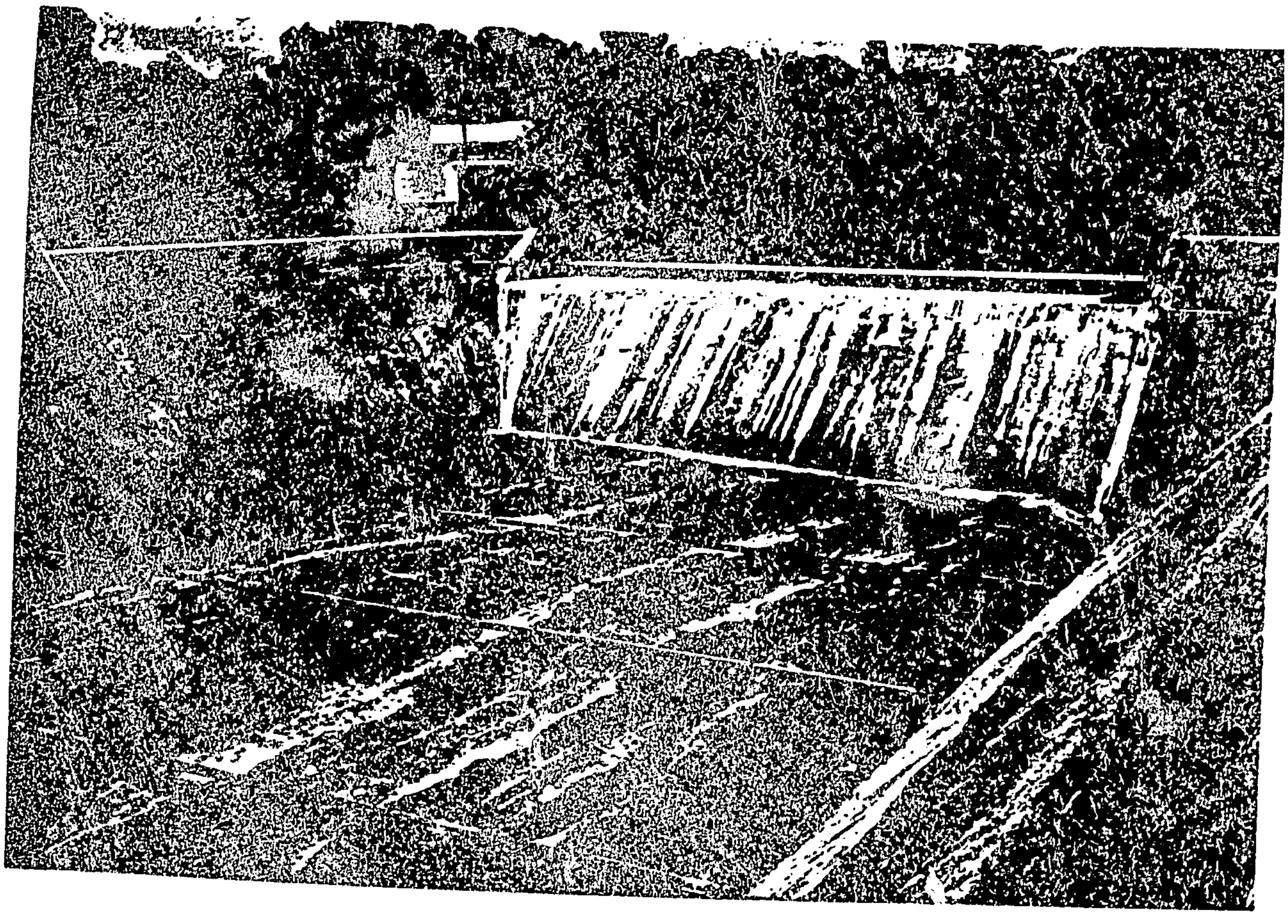


[SOURCE: Hudson, personal communication]

For sedimentation basins that are manually cleaned, a major portion of the volume is reserved for sludge accumulation between cleanings. For plants having good mixing and flocculation, the sludge layer will tend to be deeper at the inlet end than the outlet end (see Figure 7-1). Hence, a sludge storage depth tapering from 2 m at the inlet end to about 0.3 m at the outlet end is desirable (Hudson, 1981). When the sludge layer exceeds the basin's storage limitations, the settling basin should be removed from service and cleaned. To facilitate efficient drainage of the basin, the floor should slope about 10% from the side walls to the centerline, and 5 to 8% from the outlet end to the inlet end. The removal of sludge is accomplished expeditiously by first draining the basin by opening a plug valve located at the inlet end. Afterwards, the sludge remaining in the basin can be flushed to drainage with the help of either high-pressure hoses or fixed nozzles attached to the basin floor; the latter type is shown in Figure 7-8. A fixed nozzle arrangement for a sedimentation basin at Grand Rapids, Michigan called for pressure flushing for about one-half hour, through 1/8-inch holes in 2-1/2 inch pipes laid on the floor of the basin, prior to basin draining (Flinn, Weston, Bogert, 1927). If water pressure is inadequate, the flow from the adjacent basins may be used to flush the basin being cleaned by partly opening the effluent valves or gates. The entire cleaning operation, if

FIGURE 7-8

Manually-cleaned Settling Basin with Fixed Nozzles on the Floor Bottom - Latin America



[SOURCE: Okun, personal communication]

done by plant personnel or laborers familiar with the procedure, should take no longer than 12 to 18 hours. The frequency of sludge removal varies considerably depending on basin capacity, the turbidity of the raw water, and tendencies toward septicity with resulting tastes and odors or sludge flotation problems. A reasonable estimate for most plants, though, is about once every 3 to 4 months.

The installation of inclined-plate or tube settlers in sedimentation basins permits much higher surface loading rates, and hence may result in higher amounts of sludge generated in a small space when treating water of similar turbidity. Consequently, units that employ inclined-plate or tube settlers require frequent sludge removal beneath the settling modules. In such situations, hydraulically drained hoppers can be used to avoid having to drain the tank at relatively short intervals. The floors of the hoppers must be sloped no less than 55° above the horizontal, as sludge will not usually move down flatter slopes.

A hopper-bottomed horizontal flow settling tank that has been used in several treatment plants in North Carolina (Pyatt and Associates, personal communication) is shown in Figure 7-9. The design employs conventional horizontal-flow sedimentation, manual sludge removal by hydraulic discharge from hopper bottoms, and over-and-under baffles which serve to create a sludge-blanket effect above the first hopper and promote better settling in the remaining portion of the

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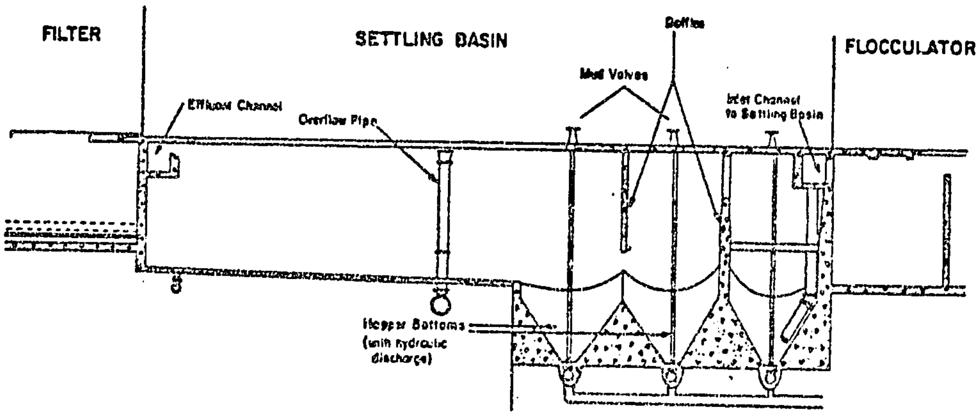
FIGURE 7-9

Hopper-bottom Settling Basin with Over-and-Under Baffles - North Carolina, USA



[SOURCE: adapted from construction drawings by Pyatt & Assoc. Consulting Engineers, Raleigh, NC]

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basin. The three hopper bottoms and two baffles are located at the inlet side of the basin where most of the floc is likely to settle out. With this type of design, the horizontal-flow portion of the basin needs only be manually cleaned about once every 6 months.

Inclined-Plate and Tube Settling

Inclined-plate and tube settling have become important components in water treatment in recent years. When installed in either upflow solid-contact reactors or horizontal-flow basins, these units can improve clarifier performance and increase the capacity of conventional clarifiers by 50 to 150 percent. Furthermore, they may also be incorporated into the design of new sedimentation basins, reducing the settling area to 1/4 to 1/6 of that required by conventional basins, and lowering construction costs by 50 to 60 percent. There are presently a number of such installations in plants in the United States, Europe, and Latin America; hence the technology for their design and construction is fairly well-developed.

A complete description of tube settling for water treatment is presented by Culp and Culp (1974), emphasizing the utilization of commercially fabricated modules in both horizontal-flow and solid-contact reactors. Design criteria and example applications are given for installing modules in existing basins. Yao (1973) gave a theoretical treatment of both inclined-tube and plate settlers, and derived several formulae for design. He concluded that the efficiency of such settlers exceeded that of conventional clarifiers under similar loadings. The design of an inclined-plate settler unit for a small package water treatment plant using Yao's approach was presented by Bhole (1981). Rao and Paramasivam (1980) summarized the existing knowledge available on tube and plate settlers to determine their applicability for developing countries. Other authors have also addressed the subject (Arboleda, 1973; Hudson, 1981; Sanks, 1978; Smethurst, 1979).

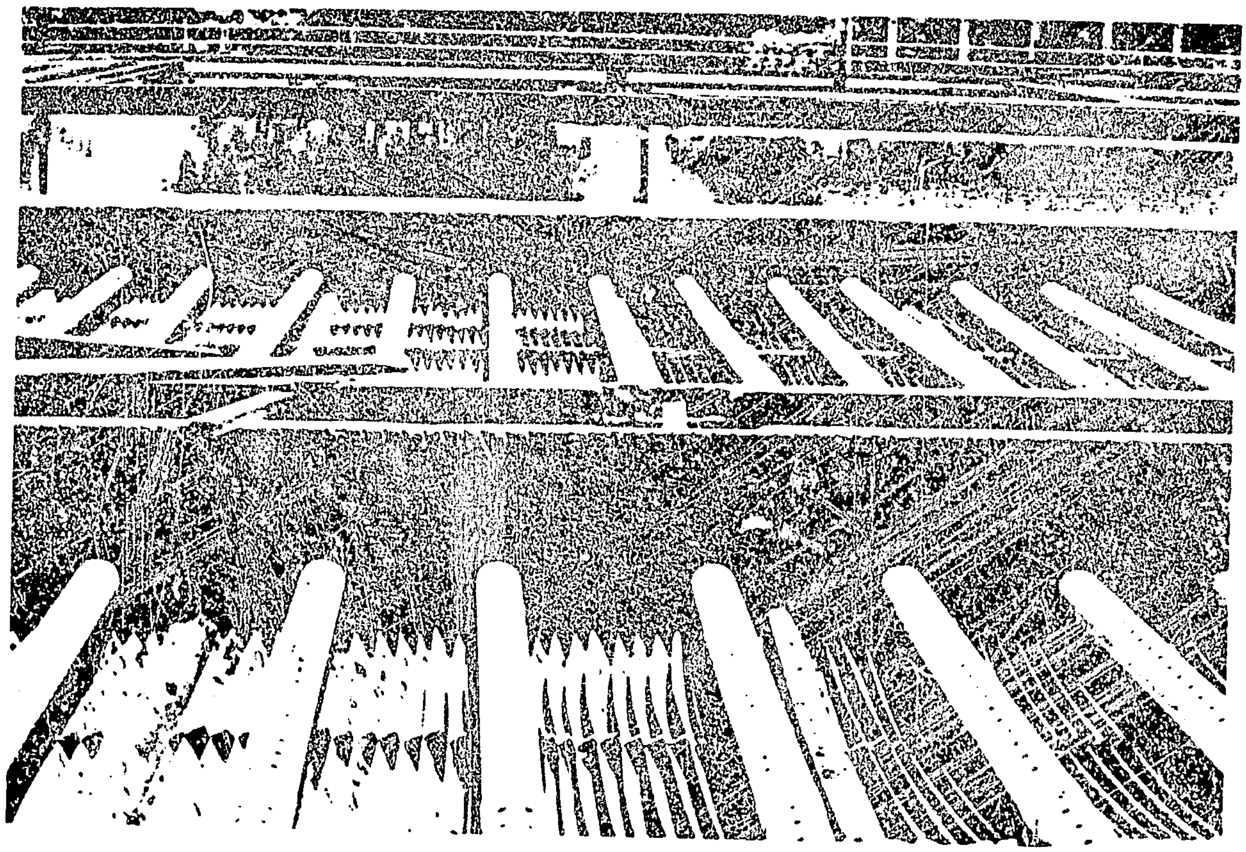
For developing countries, the use of inclined-plate or tube settling is limited, in most cases, to expanding settling basin capacity and/or improving plant effluent quality. The incorporation of settlers in the design of new plants in developing countries in order to reduce basin size and cost is usually not justified, as land is generally not restricted and low-cost labor and materials are available for construction. Moreover, when conventional sedimentation basins are installed during initial plant construction, the option remains for installing inclined-plate or tube settlers in the future, when the plant undergoes a capacity expansion, at little additional cost. If this is not done, i.e. if tube or plate settlers are installed initially, then the next plant expansion is likely to require the construction of a new settling basin. There may be certain situations, of course, where land and/or cost are the overriding constraints in design; under these circumstances such settlers should be considered. Plate settlers were installed during the capacity expansion of the treatment plant in Cali, Colombia (Medina and Hudson, 1980). A photograph of the plate settlers and perforated plastic pipe collection system is shown in Figure 7-10.

Recommended surface loading rates for horizontal-flow sedimentation basins equipped with inclined-plate or tube settlers are listed in Table 7-4 for two categories of raw water turbidity; 0-100 NTU and 100-1000 NTU. These loadings apply specifically to warm water areas (temperature nearly always above 10°C) and apply to most developing countries. For efficient self-cleaning, tubes or inclined-plates are usually arranged at an angle of 40° to 60° to the horizontal. The most suitable angle for a particular design depends on the sludge characteristics of the water being treated, usually 55° above the horizontal. The distance between parallel inclined-plates or, similarly, the diameter of settling tubes, is about 5 cm. The passageways formed by the plates, or inside the tubes, are commonly about 1 meter long.

The construction of inclined-plate or tube settlers is possible using entirely local materials and labor. For inclined-plate settlers, the individual trays can be fabricated from polyethylene (or a similar type of plastic) or

FIGURE 7-10

Inclined-plate Settlers with Perforated-plastic Pipe Outlet System at a Plant in Cali, Colombia



[SOURCE: Medina and Hudson, 1980, p. 668]

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TABLE 7-4: Loading for Horizontal-flow Settling Basins Equipped with Inclined-plate or Tube Settlers in Warm-water Areas (above 10° C)

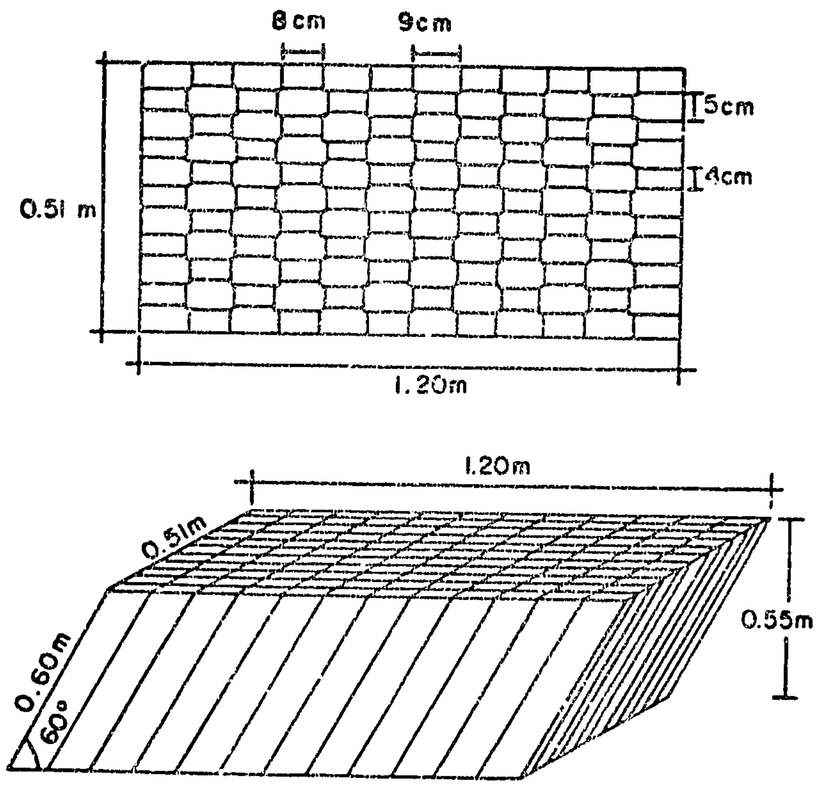
SETTLING VELOCITY BASED ON TOTAL CLARIFIER AREA (m/day)	SETTLING VELOCITY BASED ON PORTION COVERED BY TUBES (m/day)	PROBABLE EFFLUENT TURBIDITY (NTU)
	(A) Raw Water Turbidity 0 - 100 NTU	
120	140	1 - 3
120	170	1 - 5
120	230	3 - 7
170	200	1 - 5
170	230	3 - 7
	(B) Raw Water Turbidity 100 - 1000 NTU	
120	140	1 - 5
120	170	3 - 7

(SOURCE: Culp and Culp, 1970, pp. 46-47) of wood. Asbestos-cement plates should be coated with plastic or similar type of protective covering due to their susceptibility to corrosion from alum treated water. Where wood is used on low slopes, trays are commonly 30 cm apart. It may also be necessary to drain the tank for cleaning occasionally, because sludge does not readily slide down wooden trays while the basin is in service. Tube settlers, on the other hand, are easily fabricated from PVC pipes (3 to 5 cm internal diameter), which are closely packed together to form a module. In countries with indigenous plastics industries, such as Brazil, Colombia, and Mexico, commercially available tube modules which are prefabricated at the factory are suitable for larger installations. A Brazilian-built tube module is shown in Figure 7-11.

When installing inclined-plate or tube settlers in horizontal-flow sedimentation basins, it is advisable not to locate them near the inlet zone where turbulence could inhibit the effectiveness of the settlers. Furthermore, it is sometimes necessary to supplement the existing effluent collection system with additional weirs or launders so it can carry the increased loading and to allow for additional head loss through the influent flume (this head loss increases as Q^2). Figure 7-12 shows a typical tube module installation in a conventional sedimentation basin.

FIGURE 7-11

Plastic-Tube Module Fabricated in Brazil

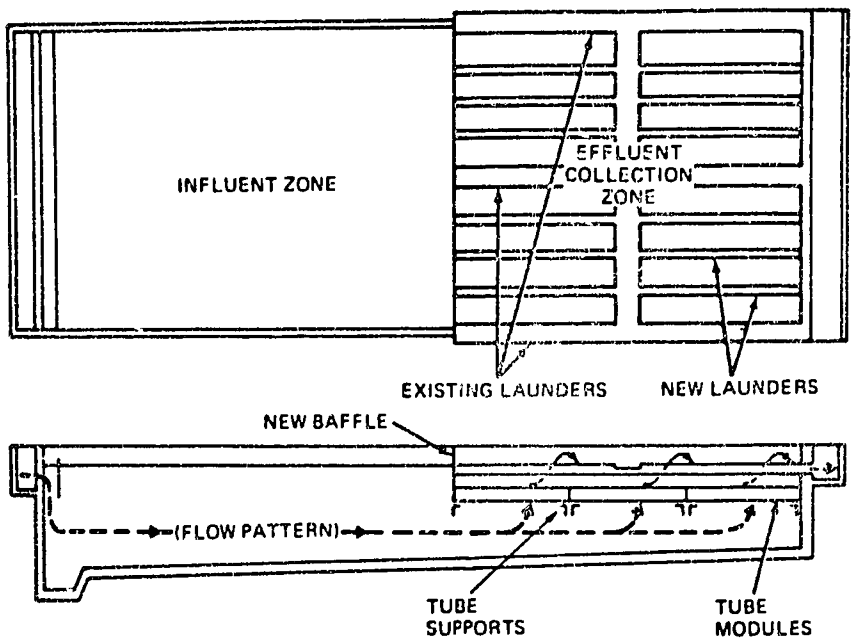


Material:	PVC
Specific gravity:	1.46
Color:	Black
Dimensions:	0.51 x 1.20 x 0.55m
Wall thickness:	1mm
Weight:	28 Kg/m^2
Cost:	Cr \$ 600/m^2 (approx. US \$ 3.50)

[SOURCE: Azevedo-Netto, 1977, p. 792]

FIGURE 7-12

Typical Tube-settler Installation in a Rectangular Basin



[SOURCE: Culp and Culp, 1970, p. 43]

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Typical hydraulic calculations for the design of tube settler modules and inclined-plate settlers in horizontal-flow basins are presented in Appendix B.

<u>Upflow Sedimentation</u>

In developing countries, the application of upflow type clarifiers should in general be limited to those areas that can comply with the following conditions: (1) relatively constant raw water quality with turbidity not exceeding 900 NTU, so as not to upset the performance of the sludge blanket; (2) plants that are designed with enough excess capacity so that the unit processes will not be overloaded; and (3) availability of skilled supervision. Also, the compact nature of upflow clarifiers may make them attractive for package plants or modular-type designs (see Chapter 9), or where land is not available to build larger horizontal-flow basins.

Upflow-type clarifiers with inclined-plate or tube settling and constructed from concrete have been designed in Brazil (Azevedo-Netto, 1977). A typical design is shown in Figure 7-13. Several advantages have been claimed for upflow-type designs:

(1) There is no need for mechanical scrapers or frequent manual cleaning (the hopper bottoms are self-cleaned by hydraulic discharge);

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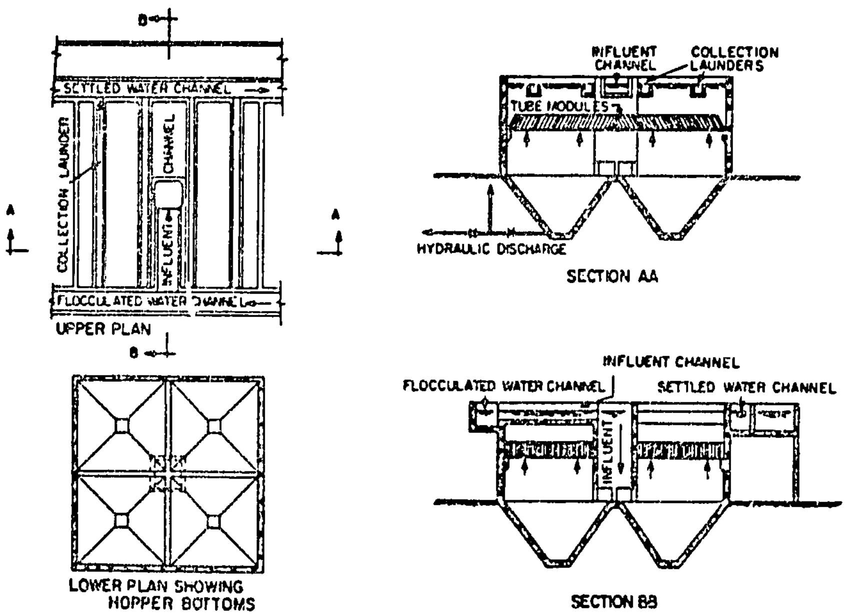
FIGURE 7-13

Concrete Upflow Clarifier with Tube Modules Constructed in Brazil



[SOURCE: Azevedo-Netto, 1977]

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(2) A smaller area is required than for horizontal-flow units;

(3) It has a better "geometry" for tube and plate settling (with tubes or plates covering the entire surface); and

(4) The volume of sludge produced is reduced because of the thickening effect of the sludge blanket.

Upflow clarifiers designed and built locally are to be preferred greatly over proprietary units which must be imported.

Upflow sedimentation is appropriate for modular treatment plant designs and package plants mainly because of the relatively small area required, especially if inclined-plate or tube settlers are also used. Typical designs which employ upflow sedimentation are shown in Chapter 8, "Upflow-Downflow Filtration"; and Chapter 9.

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VIII. FILTRATION

Filtration is a physical, chemical, and in some instances, biological process for separating suspended and colloidal impurities from water by passage through porous media. Two general types of filters are commonly used in water treatment: the slow-sand filter, and the rapid filter. A slow-sand filter consists of a layer of ungraded, fine sand through which water is filtered at a low rate, the filter being cleaned by periodically scraping a thin layer of dirty sand from the surface at intervals of several weeks to months. The sand is washed and then, after several scrapings, returned to the filter. The low rate of filtration allows the formation of an active layer of microorganisms, called the "schmutzedecke", on top of the sand bed which provides biological treatment. This layer is particularly effective in the removal of microorganisms, including pathogens, from water. A rapid filter, on the other hand, consists of a layer of graded sand, or in some instances, a layer of coarser filter media (e.g. anthracite) placed on top of a layer of sand, through which water is filtered at much higher rates, the filter being cleaned by backwashing with water.

Because of the higher filtration rates, the space requirement for a rapid filtration plant is about 20% of that required for slow-sand filters; although the latter do

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not usually require pretreatment steps (i.e. chemical treatment, rapid mixing, flocculation, and sedimentation). Table 8-1 summarizes the design criteria, and Figure 8-1 shows simplified drawings, for each type of filter. Another type of filtration scheme, not shown in Table 8-1 or Figure 8-1, but which has definite applications in developing countries, utilizes upflow filtration followed by a downflow bed, and can be an economical alternative to conventional flocculation-sedimentation-filtration schemes.

Rapid filtration plants are ubiquitous in the United States and most industrialized countries, although some countries, such as England, have found slow-sand filters to be appropriate in certain situations. Modern rapid filters are generally the most complex and costly structures to construct, operate, and maintain in water treatment plants, and are often fully automated to reduce labor costs. Filter operation is generally controlled from an operating table located directly in front of the filter, as shown in Figure 8-2, which is often automatically operated, with pushbutton standby, and the option for operation from a central control room. Such equipment enables a single operator to shut off a filter at a predetermined head loss, backwash the filter, and put it back into service by simply moving a lever or pressing a button.

In developing countries, such labor-saving automation is neither necessary nor desirable. Simple rapid filter

TABLE 8-1: General Features of Construction and Operation of Conventional Slow and Rapid Sand Fitlers

	SLOW SAND FILTERS	RAPID SAND FILTERS
Rate of filtration	2 to 5 to 10 m/day	100 to 125 to 300 m/day
Size of bed	Large, 2000 m^2	Small, 40 to 400 m^2
Depth of bed	30 cm 1f gravel, 90-110 cm of sand, usually reduced to no less than 50-80 cm by scraping	30-45 cm of gravel, 60-70 cm of sand; not reduced by washing
Size of sand	Effective size 0.25 to 0.3 mm; Uniformity coefficient 2 to 2.5 to 3	Effective size 0.45 mm and higher; Uniformity coefficient 1.5 and lower, depending on underdrainage system.
Grain size distribution of sand in filter	Unstratified	Stratified with smallest or lightest grains at top and coarsese or heaviest at bottom.
Underdrainage system	1) Split tile laterals laid in coarse stone and discharging into tile or concrete main drains. 2) Perforated pipe laterals discharging into pipe mains.	1) Perforated pipe laterals discharging into pipe mains; 2) false floor type
Loss of head	6 cm initial to 120 cm final.	30 cm initial to 240 or 275 cm final
Length of run between cleanings	20 to 30 to 60 days	12 to 24 to 72 hr
Penetration of suspended matter	Superficial	Deep
Method of cleaning	Scraping off surface layer of sand and washing and storing cleaned sand for periodic resanding of bed.	Dislodging and removing suspended matter by upward flow or backwashing which fluidizes the bed. Possible use of auxiliary scour systems.
Amount of water used in cleaning sand	0.2 to 0.6% of water filtered	1 to 4 to 6% of water filtered
Preparatory treatment of water	Generally none when raw water turbidity <50 NTU	Coagulation, flocculation, and sedimentation
Supplementary treatment of water	Chlorination	Chlorination

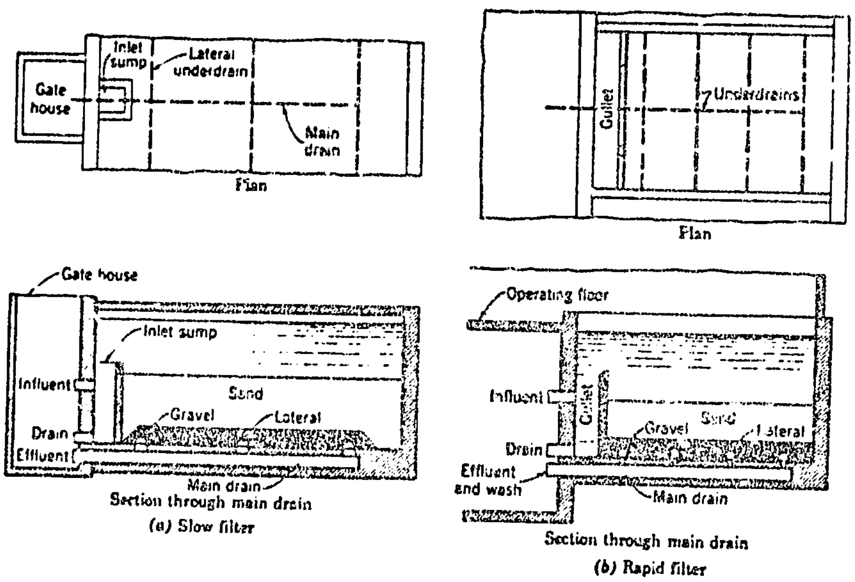
TABLE 8-1 (cont.) : General Features of Construction and Operation of Conventional Slow and Rapid Sand Fitlers

	SLOW SAND FILTERS	RAPID SAND FILTERS
Cost of construction, USA	Relatively high	Relatively low
Cost of operation	Relatively low where sand is cleaned in place	Relatively high
Depreciation cost	Relatively low	Relatively high

[SOURCE: adapted from Fair, Geyer and Okun, 1968, p. 27-4]

FIGURE 8-1

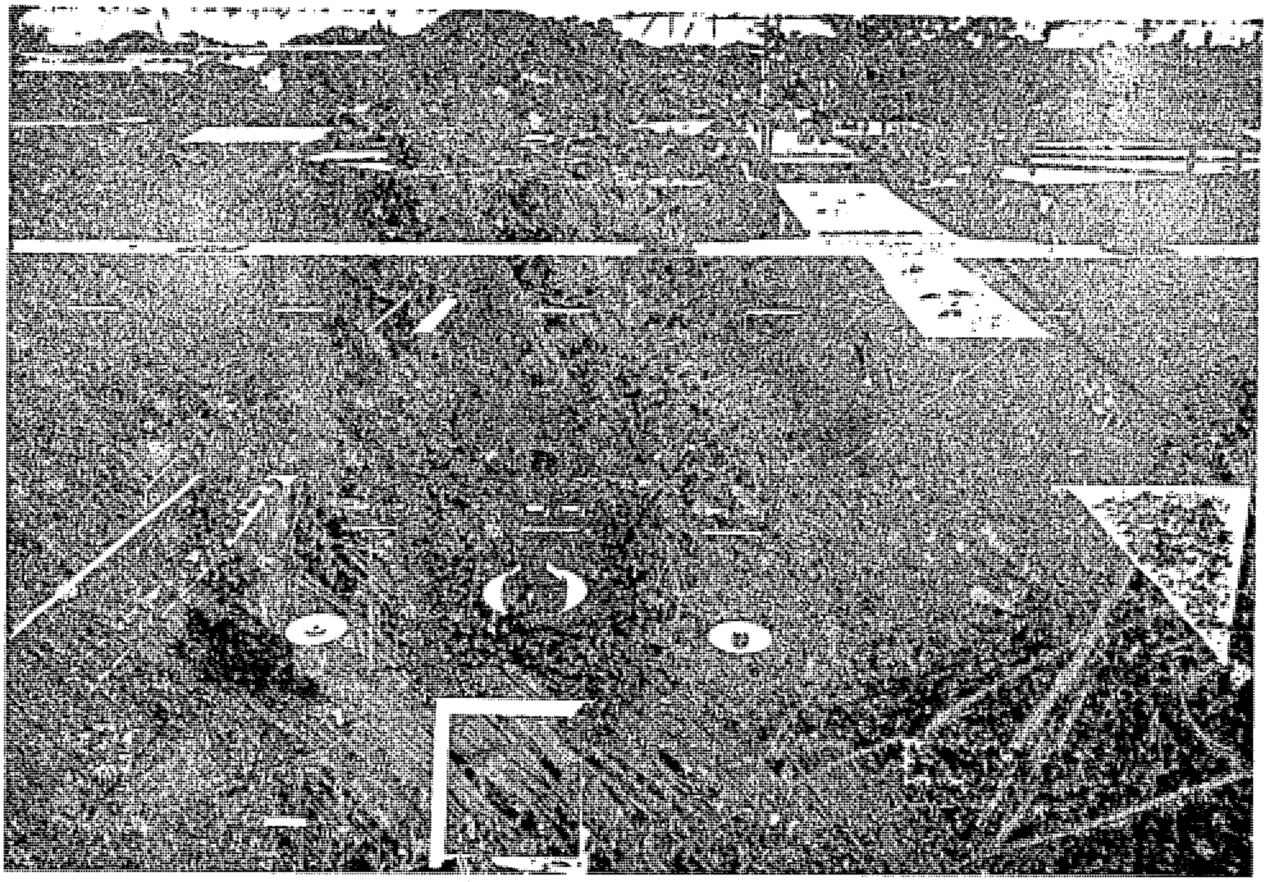
Simplified Drawings of
Slow and Rapid Filters



SOURCE: adapted from Fair, Geyer, and Okun, 1968]

FIGURE 8-2

"Labor-saving" Filter Operating Table at a Large Water Treatment Plant in Asia



[SOURCE: Okun, personal communication]

designs that employ manual controls, such as the hand-operated valve shown in Figure 8-3, and that eliminate, to the extent possible, unessential mechanical equipment and instrumentation are much to be preferred. Also, when conditions are suitable, slow-sand filters or upflow-downflow type filters can provide simple solutions at low cost.

This chapter emphasizes the design and operation of simple types of rapid filters, including the upflow-downflow types. Slow-sand filtration, which may be the most practical technology for treating some surface water supplies in developing countries, is examined to a lesser extent as other manuals and publications on this subject are widely available. Information sources on slow-sand filtration are given near the end of this chapter.

Rapid Filtration

Rapid filters can be classified in various ways. They may be classified according to (1) the type of filter media employed; (2) the type of filter rate control system employed; (3) the direction of flow through the bed; or (4) whether they operate under gravity (free-surface) or pressure. In general, pressure filters are not well suited for developing countries because they generally need to be imported, they require skilled operation, and their parts are not accessible for easy maintenance.

FIGURE 8-3

Hand-operated Valve for Washing a Filter at a Plant in India

[SOURCE: Okun, personal communication]

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Constant-rate filtration and declining-rate filtration are the two basic types of control systems. Constant-rate filters are equipped with a rate-control device in the effluent line, which provides an adjustable resistance to the water flow, and compensate for the increasing head loss in the rate controllers, and hence their operation is much simpler. Table 8-2 compares the characteristics of declining-rate with constant-rate systems. The design of declining-rate filters is discussed later in the chapter.

The design variables for rapid filtration, include (1) dual-media filter beds, (2) filter bottoms and underdrains, (3) backwashing arrangements, (4) auxiliary scour wash systems, and (5) declining-rate filtration. These are described below. Direct filtration is also reviewed for its suitability for developing countries. Additional, more detailed information on the design of conventional rapid filters may be found in several standard references (AWWA, 1971; Arboleda, 1973; Fair, Geyer, & Okun, 1968; Hudson, 1981; Sanks, 1979).

Dual-Media Filters

Sand has been used traditionally as the filter medium in water treatment plants because of its wide availability, low cost, and the satisfactory results that it has given. Sand filters remain the predominant method of filtration in developing countries. On the other hand, dual-media beds have gradually replaced the sand in rapid filters during the

TABLE 8-2: Characteristics of Filtration Systems

	DECLINING RATE	CONSTANT RATE
1.	Hydraulic master control of all filters simultaneously is inherent.	Requires central set point station and powered signals transmitted to all filters.
2.	Tendency toward terminal breakthroughs is substantially reduced because both terminal rate and head loss are reduced. Water quality is thereby improved.	Subject to severe terminal breakthroughs. Water quality impaired.
3.	For the same average rate of filtration, filter runs are longer.	Shorter filter runs.
4.	System is not dependent on the functioning of a number of costly mechanical devices.	Numerous devices required.
5.	Need for maintenance of rate controllers and master control devices is eliminated.	Continuous maintenance of rate controllers and master control equipment.
6.	Use of a fixed restraint on filtration rates makes it difficult for the operator to run, filters improperly.	Operator can easily prop or tie controllers into wide-open position.
7.	Operation is controlled from a single differential head measurement, rather than from a measurement on each filter.	Requires head loss measurement for each filter to attain control.
8.	Individual filter rate-of-flow and loss-of-head gages are not required.	Rate-of-flow and loss-of-head gages required.
9.	Once the operator is accustomed to the system, he finds it simpler to use than than the traditional control scheme; the filters accept whatever flow is asked of them.	Flow through plant must be regulated both at inlet and at filters.
10.	The system can be designed to minimize surge effects.	Surge problems unavoidable.

past 20 years (Culp and Culp, 1974). The dual-media filter is generally composed of a coarse coal upper layer (specific gravity of 1.45 to 1.55) which acts as a roughing filter to reduce the load of particulates to the lower sand layer (specific gravity of 2.65). Because of the different specific gravities of the two materials, the two layers retain their relative positions after backwashing, although the coarse-sized material mixes with sand near the interface instead of being stratified after backwashing.

Dual-media filters possess several distinct advantages over conventional sand filters: (1) higher filtration rates are allowed (10 to 15 m/hr) than those for conventional filters, resulting in a reduction in the total filter area required for a given design rate of flow; (2) more impurities removed from the water are retained in the filter bed, thereby improving filter effluent quality; (3) the length of filter run is increased before terminal head loss is reached; and (4) by the conversion to dual-media beds, the capacity of existing sand filters can be easily increased at low cost.

This last advantage may be exceedingly beneficial to those communities in developing countries that are burdened with overloaded and inefficient treatment plants. The unique characteristics of dual-media beds are such that they can be incorporated into an existing filtration system without change in plant structure or method of operation.

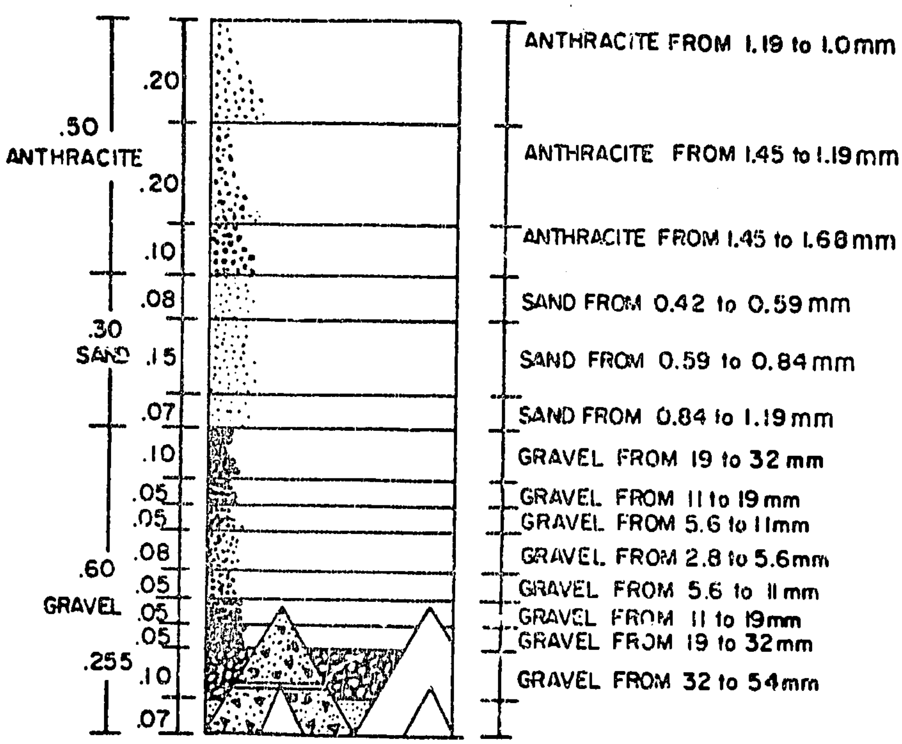
Also, a wide variety of unconventional filter media, such as crushed coconut shells or bituminous coal that are indigenous and low in cost, are suitable as coarse material for the upper layer. The cost for converting plants from single media sand filters to high-rate dual-media filters was estimated in India at only \$0.80 per m^3/day of plant capacity, or \$0.40 per m^3/day of increased capacity (Ranade and Gadgil, 1981). These estimates included the cost of modification in the influent and underdrain systems, and the cost of placing new media. Experiments conducted in Wisconsin (Culp and Culp, 1974) compared "coal capped" sand filters (i.e. where 6 inches of sand is removed from a filter bed and replaced by 6 inches of anthracite coal) and sand alone under various raw water conditions. The capped filters were operated at 7.2 m/hr and the sand at 4.8 m/hr. The result showed that the coal capped filters performed better, and gave up to a 10 to 1 improvement in filter runs for the worst raw water conditions.

The terms "effective size" and "uniformity coefficient" are used in defining filter media. The effective size (E.S.; P_{10}) is the particle size, in millimeters, such that 10% of the particles by weight are smaller and 90% are larger. The size distribution is characterized by the uniformity coefficient (UC), the ratio of the P_{60} to P_{10} sizes. These two parameters are determined for a particular filter material by standard sieve analyses (Cox, 1964; Fair, Geyer, & Okun, 1968). Most filters in use today contain sand with an effective size of 0.4 to 0.8 mm as the under layer, topped by a layer of coarse materials with an effective size of 1 to 1.6 mm (IRC, 1981b). The uniformity coefficient for each layer is usually between 1.3 and 1.7. The depth of the filter bed should be large enough to prevent a significant amount of impurities from reaching the filtered water outlet, and is best determined by pilot filter tests, especially if unconventional filter media with largely unknown physical properties are being considered. Typically, dual-media filters consist of coarse media at a depth of 40 to 75 cm and a sand layer about 15 to 30 cm deep. Figure 8-4 shows a typical cross section through an anthracite-sand-gravel filter bed with graded layers for each filter medium and a reverse-gradation scheme for the supporting gravel.

In places where sand cannot be purchased by specification and locally available sand is not properly sized, a simple procedure for grading filter sand may be used (Cox, 1964). The sand is screened through a coarse screen to remove foreign matter. It is then placed in a filter box and backwashed at higher than normal rates, allowing the undesirable fine sand to be wasted. The remaining fine sand that is undesirable is removed from the surface using hand tools.

FIGURE 8-4

Typical Dual-Media Filter Bed
(cross-section)



NOTE: ALL DIMENSIONS IN METERS UNLESS OTHERWISE NOTED

[SOURCE: adapted from CEPIS, Vol. 2, plan no. 37]

Indigenous coals and other unconventional filter media are available and have been used in many countries, such as Brazil, Colombia, and India. They are now under investigation in Korea, and have been used in Japan. To the maximum extent feasible, indigenous materials should be used. Bituminous coal with a specific gravity of 1.45 has performed well in Brazil. The continuing losses are greater than with anthracite, but its use has been very cost-effective.

Pilot plant and full-scale plant studies on unconventional filter media have been conducted in India (Ranade and Gagdil, 1981). The lack of anthracite coal in India, which is commonly used in the United States as the coarse layer in dual-media filters, has prompted researchers to investigate various other materials such as high-grade bituminous coal (Paramasivam et al., 1973), crushed coconut shell (Kardile, 1972), berry seeds (Bhole and Nashikar, 1974), and kernels of stone fruits such as apricots (Ranade and Agrawal, 1974). All of these materials were found to be suitable as coarse filter media, but high grade bituminous coal possessed the best overall characteristics in regards to cost, availability, and filtration properties. The physical specifications for a dual-media filter consisting of a layer of bituminous coal (specific gravity of 1.2) over a layer of sand (specific gravity of 2.65) are given in Table 8-3. The physical specifications for a dual-media

TABLE 8-3: Characteristics of Dual-Media Filter Consisting of Bituminous Coal and Sand

ITEM	COAL	SAND
Size Range	0.85 - 1.6 mm	0.55 - 0.9 mm
Effective Size	1.0 mm	0.6 mm
Uniformity Coefficient	1.3 - 1.5	1.3 - 1.5
Specific Gravity	1.2	2.65

[SOURCE: adapted from Ranade and Gadgil, 1981, p. 83] filter consisting of crushed coconut shells over sand are given in Table 8-4.

Several treatment plants have been constructed in India with crushed coconut shells as the coarse filter medium (Kardile, 1981). The dual-media filters in the Ramtek plant (see section on "Upflow-Downflow Filters") consist of a 30 cm layer of coconut shells (average size = 1 to 2 mm) over 50 cm layer of fine sand (E.S. = 0.45-0.55 mm); and have been operating for nearly a decade without any deterioration in filtrate quality.

A dual-cum-mixed media filter (composed of a coarse layer of crushed coconut shells and mixed media of 30% sand and 70% boiler clinker) was tested against a dual-media filter (composed of crushed coconut shells and sand) to compare filtration performance (Bhole and Rahate, 1977). After a 24-hour run, it was found that the dual-cum-mixed media filter gave almost half the head loss of the dual-media filter while still maintaining comparable turbidity removal. Hence, this type of filter was considered more economical because of the lower head loss as well as the lower cost clinker media. The characteristics of the three types of media (sand, coconut shells, and boiler clinker) that comprise this filter bed are presented in Table 8-5.

A study conducted over a period of one year in India (Rao, 1981) has shown that selected crushed stone can be

TABLE 8-4: Characteristics of Dual-media Filter Consisting of Crushed-Coconut Shells and Sand

ITEM	CRUSHED COCCNUT SHELL	SAND
Size Range	0.81 - 2.1 mm	0.5 - 0.81 mm
Effective Size	0.80 mm	0.52 mm
Uniformity Coefficient	1.2	1.3
Specific Gravity	1.4	2.65
Depth	37.5 cm	37.5 cm

[SOURCE: adapted from Nashikkar, Bhole and Paramasivam, 1976, p. 15]

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TABLE 8-5: Characteristics of Mixed-media Filter Consisting of Crushed-coconut Shells, Boiler-Clinker, and Sand

ITEM	CRUSHED COCONUT SHELL	BOILER-CLINKER	SAND
Size	1 - 2 mm	1 - 2 mm	0.5-0.85 mm
E. S.	1.1 mm	1.2 mm	0.54 mm
U. C.	1.4	1.5	1.3
S. G.	1.4	1.9	2.65
Depth	20 cm	20 cm[a]	20 cm[a]

[a]sand 30%, boiler-clinker 70%

[SOURCE: adapted from Bhole and Rahate, 1977, pp. 31-32] used as a filter medium instead of sand. Crushed stone is easily prepared from stone dust which is a waste product at quarries using stone crushers. Both fine grain (E.S. of 0.47 mm and a U.C. of 1.5) and coarse grain (E.S. of 0.7 mm and a U.C. of 1.3) crushed stone filter media were tested against a sand filter under filtration rates ranging from 4.7 m/hr to 9.8 m/hr. With both grain sizes, the performance of the crushed stone medium was better than that of the sand medium with respect to (1) turbidity removal, (2) bacterial removal, and (3) length of filter run. In places where good quality sand is not available, or must be transported from long distances at high cost, crushed stone may provide an economical alternative; but crushed limestone should not be used, as it may dissolve.

Filter gravel supports the filter media and aids in the distribution of backwash flow from the underdrain system. Gravel should consist of rounded silica stones with an average specific gravity of not less than 2.5. It should be free from clay, sand, and organic impurities of any kind. The depth and grading of gravel are related to the type of filter underdrain system used. A reverse gravel gradation, such as that used for the Teepee filter bottom in Figure 8-4, has been found to be safe against movement of the top gravel layer (Hudson, 1981). Less gravel is required with prefabricated filter bottoms, such as the Leopold or Wheeler units.

Filter Bottom and Underdrains

The two major requirements of the filter underdrainage system are the support of the filter bed without loss of media and the uniform distribution of the washwater across the entire filter bed (Culp and Culp, 1974). However, for the design of interfilter-washing units, the head loss is limited to only 20 to 30 cm, possibly at some sacrifice in uniformity of washwater distribution. However, the velocities are low enough in the plenum so that the slight pressure variations should not adversely affect backwash performance.

In many instances, bottoms can be either locally produced, reinforced concrete slabs with plastic or glass tube orifices, or simple perforated-pipe lateral systems. A locally precast reinforced concrete filter bottom with a low orifice head loss system called the "Teepee" has been adapted from California for use in interfilter washing filtration systems in Latin America and in the Philippines (Arboleda, 1973). Because of its angular shape, this system was named after the American Indian Teepee, which was a cone-shaped tent. The Teepee filter bottom is illustrated in Figures 8-5 and 8-6; the former showing cross-sectional and isometric views, and the latter showing the installation of this filter bottom in a filter box. The angle-shaped beams that comprise the filter bottom are supported at each end by the sidewalls of the filter

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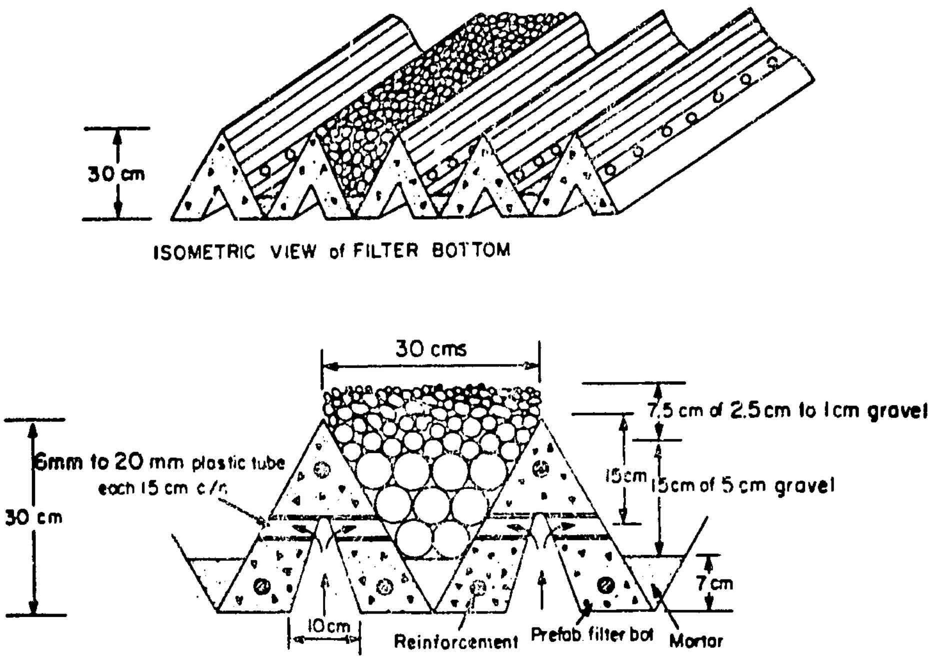
FIGURE 8-5

"Teepee" Filter Bottom used in
Latin American Filtration Plants



[SOURCE: adapted from Arboleda, 1973, p. 397]

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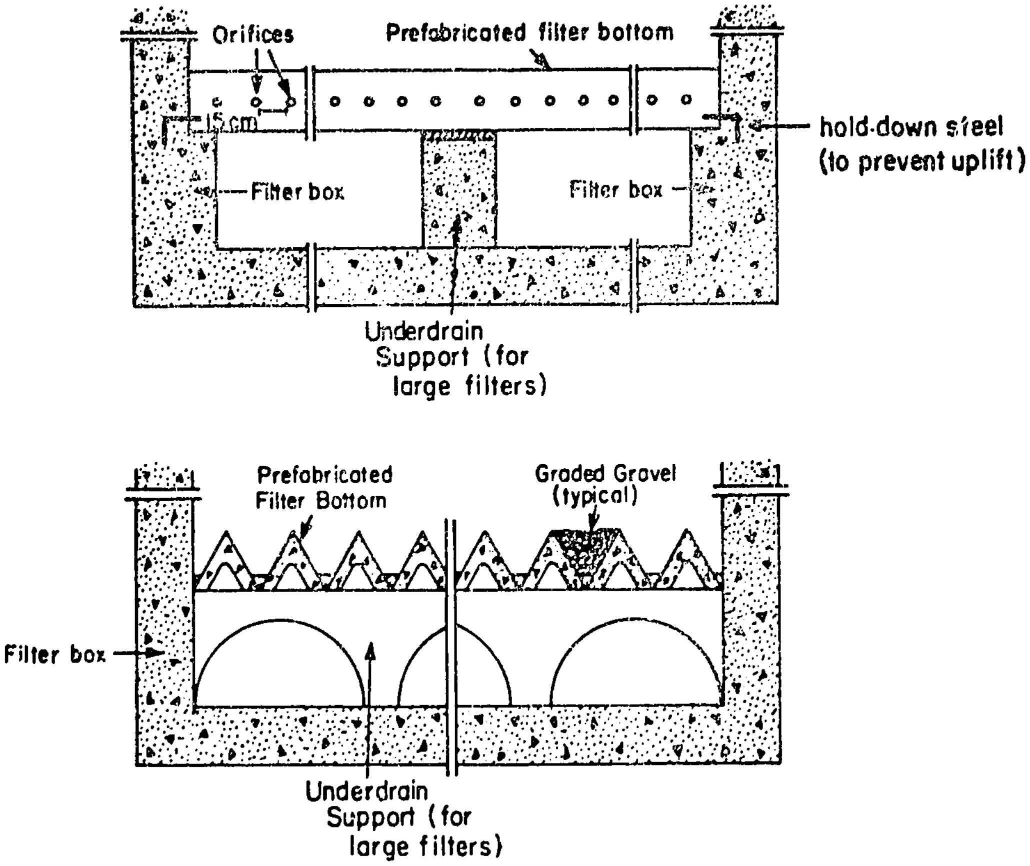
FIGURE 8-6

"Teepee" Filter Bottom Placed in the Filter Cell



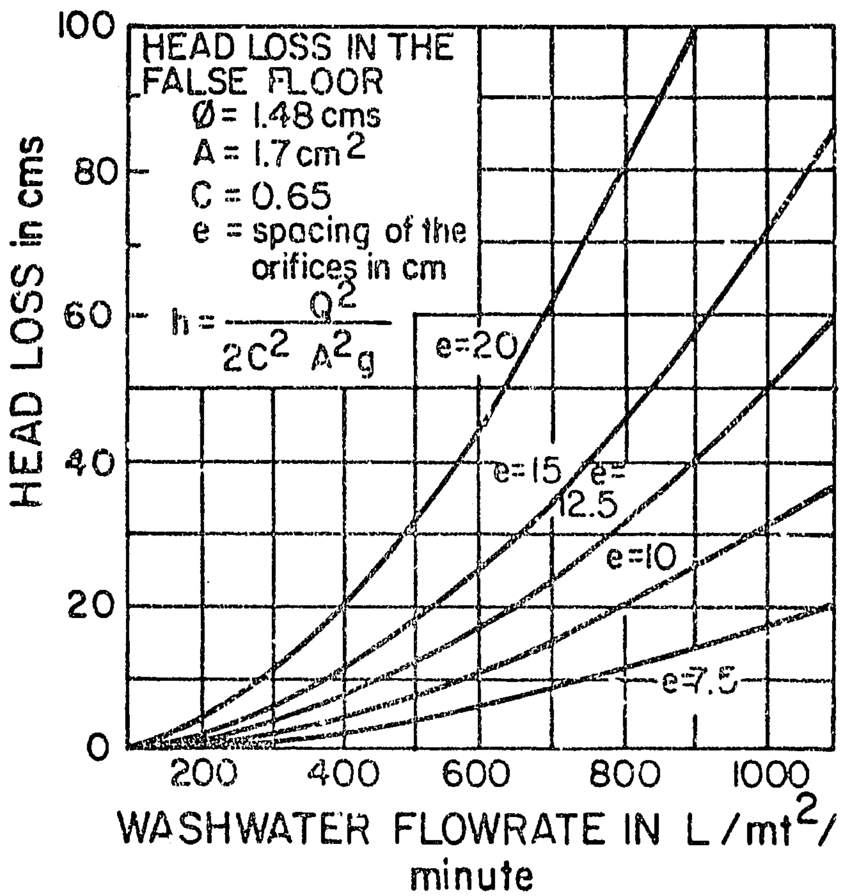
[SOURCE: Arboleda, 1973, p. 397]

box. Plastic tubes of 6 mm to 20 mm diameter are inserted along the concrete beams at 10 to 25 cm on center to form the orifices. The beams are joined together with mortar to prevent loss of filter media and to waterproof the joints. Adequate space is provided between the beams so that 3 rows of 4 cm gravel below a graded layer of 2.5 cm to 1 cm gravel can be placed there. When possible, porcelain balls or hollow plastic spheres filled with mortar may replace gravel in order to provide more uniform flow distribution (similar to the Wheeler underdrain system which uses porcelain balls in recessed pyramids underneath a smaller gravel layer). the spacing of the orifices along the concrete beams dictates the head loss in the system, as indicated in the graph of Figure 8-7, which may be used for design purposes. For example, a washwater flow rate of 36 m/hr (600 l/m^2/minute) and an orifice spacing of 15 cm would produce a head loss of 25 cm with this type of underdrain system.

The perforated pipe lateral system consists of a central manifold pipe to which are attached a series of lateral pipes with orifices to distribute washwater or to collect the filtered water, as shown in Figure 8-8. Two points are important to note in their design: (1) the losses through the orifices are kept comparatively high (about 5 meters) to maintain uniform distribution of backwash water; and (2) the highest head losses occur at the furthermost orifice from the inlet end during backwashing.

FIGURE 8-7

Head Loss in the "Teepee" Filter Bottom for Different Flowrates



[SOURCE: Arboleda, 1973, p. 397]

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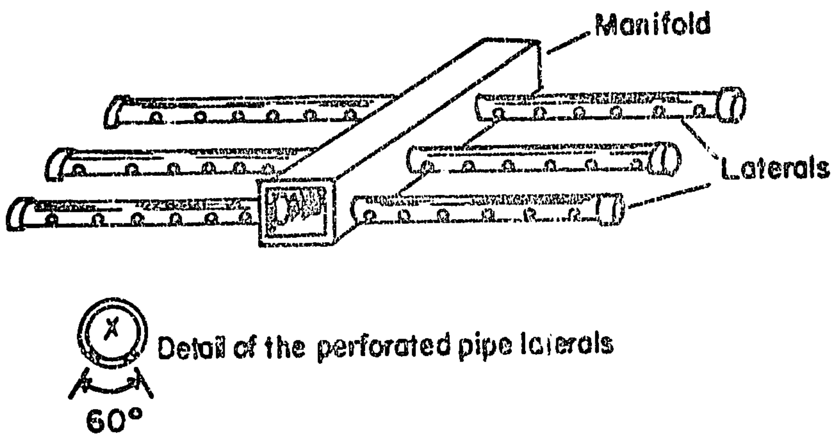
FIGURE 8-8

Main and Lateral Underdrain System



[SOURCE: adapted from Arboleda, 1973, p. 394]

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Fair, Geyer, and Okun (1968) suggest the following guidelines to pipe-lateral and underdrain design:

1) Ratio of area of orifice to area of bed served: .0015:1 to .005:1

2) Ratio of area of lateral to area of orifices served: 2:1 to 4:1

3) Ratio of area of manifold to area of laterals served: 1.5:1 to 3:1

4) Diameter of orifices: 0.6 cm to 2 cm

5) Spacing of orifices: 7.5 cm to 30 cm on centers

6) Spacing of laterals: about the same as spacing of orifices.

A number of different types of proprietary underdrain systems are available (e.g. Leopold and Wheeler filter bottoms) that may be suitable for use in larger plants in developing countries, if they can be manufactured within the country. The Leopold bottom consists of vitrified clay channels, with orifices that distribute water evenly along the entire length of the channel. Types of proprietary filter bottoms that call for strainers or false bottoms or porous plates are not generally recommended because of clogging problems and, in some cases, ruptures of the falsefloor (Hudson, 1981).

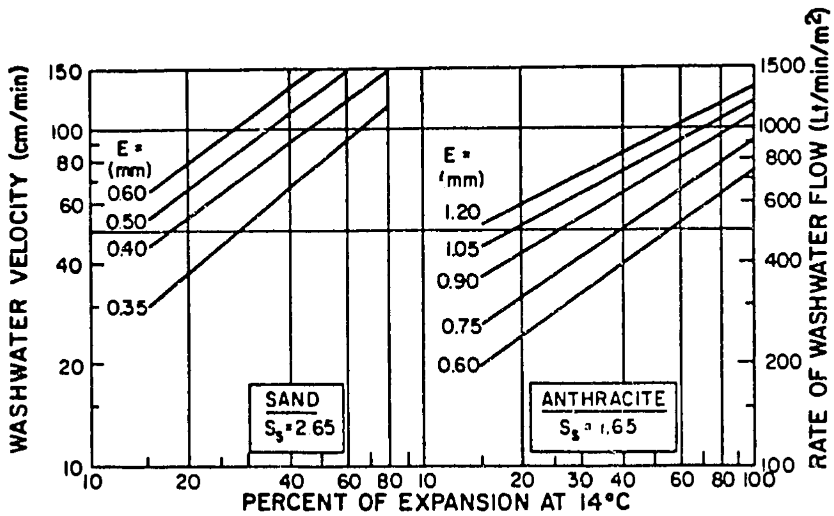
Backwashing Arrangements

The purpose of backwashing is to remove the suspended material that has been deposited in the filter bed during the filtration cycle. When a filter is backwashed, an upward flow is introduced at a rate sufficient to fluidize the filter media, and to allow the accumulated contaminants to be carried away by the washwater to waste. Theoretically, beds can be adequately cleaned when the entire bed is expanded. The percent expansion that accompanies this is a function of the size, specific gravity of the media, and viscosity of water.

Rates of backwash need to be high enough to fluidize all the filter media, but no higher. Although such rates can be calculated, they are easily determined in the field through the use of a probe rod that can reach the top of the gravel. The backwash rate required to expand the entire bed can be calculated easily by measuring the rate of rise of backwash water in the filter at that backwash setting. Excessive rates should be avoided as they waste valuable water, may disturb the supporting gravel, and are less effective in washing because the sand grains are separated further than necessary. The graph in Figure 8-9 is useful for sizing the height of washwater gullets and indicates the percent of expansion that can be expected for different sizes of sand and anthracite when using a given flow rate (or velocity) of washwater at a temperature of 14°C. The minimum expansion that will completely fluidize the media should be used. Table 8-6 can be used to adjust the values obtained from Figure 8-9 for any washwater temperature.

FIGURE 8-9

Backwash Velocities and Flowrates for Sand and Anthracite
for Different Expansion Rates at 14°C



[SOURCE: Arboleda, 1973, p. 380]

TABLE 8-6: Effect of Temperature on Required Backwash Rate for Equal Expansion

	RATE AT 14°C = 1.00						
Temperature °C	4	6	8	10	12	14	16
Backwash Rate	0.75	0.80	0.85	0.90	0.95	1.00	1.05
Temperature °C		16	18	20	22	24	26
Backwash Rate		1.05	1.11	1.16	1.22	1.28	1.35

The required rate of washwater flow for equal expansion of the filter bed is 35% greater at a temperature of 26°C than at 14°C. Similarly, the washwater rate at 4°C is 25% less than that at 14°C. The time required for backwashing varies from 3 to 15 minutes, but may be substantially reduced by incorporating auxiliary-scour wash systems in the filter units to provide quicker and more thorough cleaning of the filter media.

In order to determine the total head required to provide the design rate of flow of washwater, it is necessary to compute the head losses in the system during backwashing; including the losses attributed to the filter media, supporting gravel, underdrain system, and appurtenances (e.g. washwater pipelines and backwash-rate controllers). Most of the head loss occurs in the underdrain system (1 to 4.5 meters), although interfilter-washing units are designed with relatively low head losses in the underdrain (20 to 30 cm). A graph for determining head losses in prefabricated concrete underdrain systems is given in a previous section ("Filter Bottoms and Underdrains"). Head loss through the gravel is small, generally less than 8 cm. Head loss through the fluidized filter bed may be calculated from the following equation:

$$h = D\ (1-f)\ (p-1)$$

where

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where

h = head loss across the fluidized bed (m)

D = unexpanded bed depth (m)

f = porosity of unexpanded bed (dimensionless)

p = specific gravity of the filter medium (dimensionless) assuming specific gravity of water is 1.0

The head loss is the weight of the expanded media in the water, which is represented by the above equation. When the entire bed is expanded, the head loss becomes independent of the percent expansion, so that any washwater rate greater than that necessary to expand the bed, separates the grains to no useful purpose. A useful guideline is to establish the rate which will assure complete bed expansion. This does not vary much where temperature variations are small.

Three types of backwash arrangements that are suitable for developing countries are:

1) elevated washwater tanks;

2) taking washwater from a high-pressure distribution system tank; and

3) interfilter-washing units, i.e. an arrangement whereby one filter is backwashed with the effluent from other units.

Large washwater pumps which take suction from the filtered-water clear well and must be sized to supply a backwash rate of at least 36 m/hour are not recommended since they are costly, must be continuously maintained, and depend on a reliable power source.

An elevated washwater tank should have sufficient capacity to wash two filters, for at least 8 minutes each at the designed backwash flow rate. Larger tanks are needed when several filter units are used, so that they may be washed in sequence without the opportunity to refill the tank. The following equation may be used to calculate the required volume of the washwater tank (Arboleda, 1973):

$$V_c = A(t_e q_a + t'_e q'_a)\ n^{1/3}$$

where

V_c = volume of washwater tank (m^3)

A = are of filters (m^2)

t_e = time for surface washing (hour)

t'_e = time for backwashing (hour

q_a = surface wash flow rate (m/hour)

q'_a = washwater flow rate (m/hour)

n = number of filters

Small pumps are used to fill the washwater tank during intervals between successive backwashing. Three pumps are usually provided, with one serving as a reserve unit. The total capacity of the operating pumps should be about 10 to 20% of the washwater rate (IRC, 1981b). The bottom of the tank should be high enough above the washwater gullet to provide the desired washwater flow rate, as determined from an analysis of the head losses in the system. This distance normally ranges from 4 to 6 meters (IRC, 1981b). Washwater tanks should each be equipped with an overflow pipe, drain

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valve, air vent, vortex-breaking baffle, and manually-operated washwater regulating valve, as shown in Figure 8-10. Tanks are constructed of steel or concrete, depending on local costs and availability of these materials.

When the distribution system, with its high pressure, is near the treatment plant, washwater may be taken from the distribution system. Such an arrangement eliminates the need for a separate washwater tank and pumps to fill the tank. However, it is necessary to reduce the pressure because distribution system pressures are generally higher than those needed for effective backwashing and, if not controlled, may blow out the filter. One option is to feed water from the distribution system into an elevated washwater storage tank during periods of low demand. The washwater can then be fed by gravity to the filters when backwashing is needed.

Interfilter washing systems are virtually free from ancillary backwash equipment such as washwater tanks, pumps, pipe galleries, and washwater rate controllers. The washwater and pressure head for backwashing a cell are obtained from companion cells that are connected in parallel through a common underdrain system, as shown in Figure 8-11. A cell is backwashed by closing the inlet and opening the drainage outlet of the cell. The water level in the cell is thus lowered, creating a positive head (H_b) which reverses

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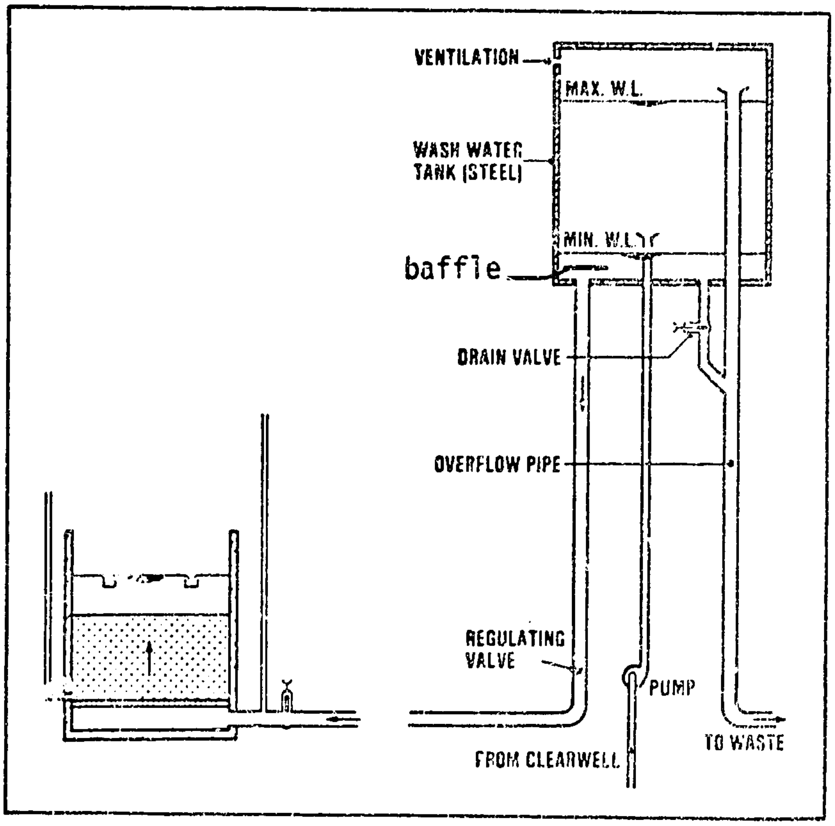
FIGURE 8-10

Washwater Tank Arrangement



[SOURCE: IRC, 1981b, p. 283]

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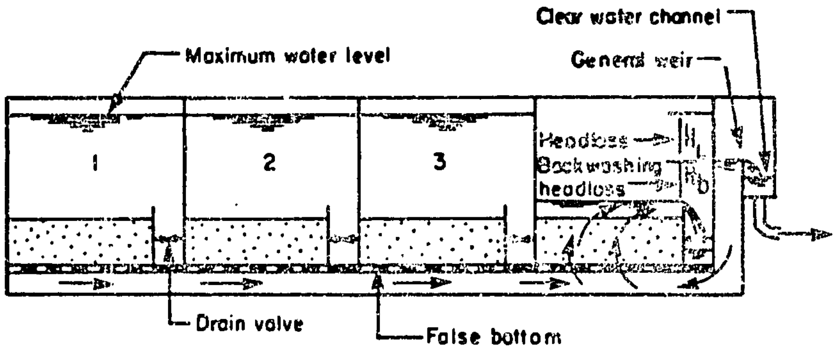
FIGURE 8-11

Backwashing of One Filter with the Flow of the Others



[SOURCE: Arboleda, 1973, p. 385]

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the direction of flow through the filter bed and initiates the backwash cycle. After washing, the drain is closed and the inlet opened. The cell then resumes its filtration cycle.

The available head for backwashing, H_b, is the difference in elevation between the effluent weir and the gullet lip in the filter cell. The required value of H_b to expand the filter bed is the sum of the head loss in the underdrain and pipe system and the head required to keep the filter media in suspension. By increasing the depth of the water over the filter beds (about 1.5 to 2.5 meters), limiting the head loss in the underdrain system (about 20 to 30 cm), interconnecting the underdrain systems, and using dual-media filter beds, the backwashing head (H_b) will be sufficient to produce the desired expansion rates. The design and operation of interfilter washing units are discussed in the following section of this chapter.

Washwater may be collected and removed from the filter cell by either: (1) a system of troughs and gullets, or (2) only gullets. Although washwater troughs are used cxtensively in the United States mainly because of tradition, there has been no evidence to indicate that they measurably improve the backwashing process. In fact, gullets have performed admirably as the sole washwater collection system in a number of plants in the United States (Hudson, 1981). Center and side gullet designs are shown in

Figure 8-12. In constructing a gullet wall it is convenient to bevel the edge so that the flat top surface is narrow and can be leveled accurately by grinding. Flat bottomed gullets may be designed with the help of the following formulae and the nomenclature diagrams presented in Figure 8-13 (Fair, Geyer, and Okun, 1968):

$H = [h^2 + (2Q^2/gb^2h)]^{1/2}$ for submerged discharge (8-3)

$H = 1.73h = (0.4Q/b)^{2/3}$ for free discharge (8-4)

where

H = depth at upstream end (m)

h = depth at downstream end (m)

Q = rate of discharge (m^3/sec)

g = gravity constant (9.81 m/sec^2)

b = width of channel (m)

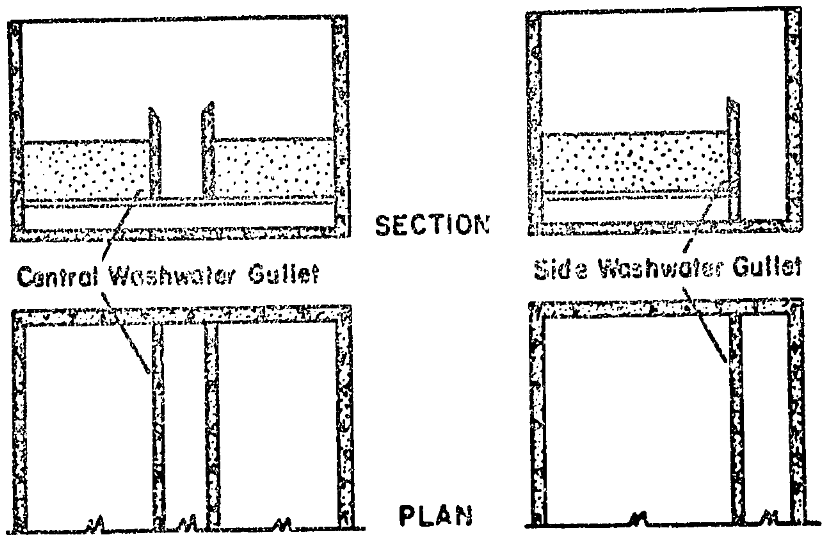
In most designs, the depth at the upstream end (H) is known, and the width of channel (b) is solved by successive trial and error. Generally, it is good to provide an additional factor of safety for height, in case larger than anticipated backwash rates are used.

Auxiliary-Sccur Wash Systems

Auxiliary-scour is used to assist in cleaning the filter media and to prevent mud ball formation and filter cracking. Air scouring and surface wash are two basic types of auxiliary-scour wash systems. The fixed-grid types of surface-wash systems are best suited for developing countries because of their simplicity in design and lack of

FIGURE 8-12

Arrangements for Washwater Gullets



[SOURCE: Arboleda, 1973, p. 386]

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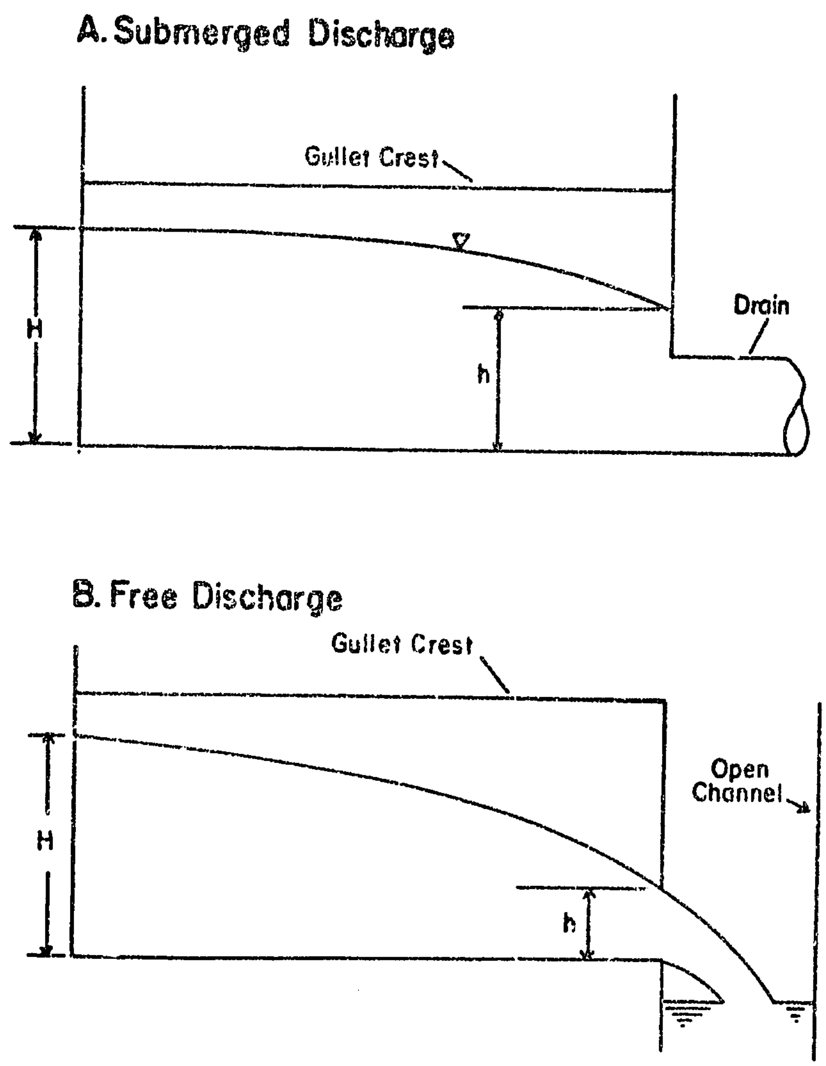
FIGURE 8-13

Nomenclature Diagrams for Side Weir or Gullet Design



[SOURCE: adapted from Hudson, 1981, p. 234]

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moving parts. Baylis designed such a system, consisting of a grid of distributing pipes about 10 cm above the top surface of the bed (Cox, 1964). Plastic or metal caps with five 6-mm holes are spaced at about 60 to 75 cm center to center. Water pressures of 70-200 KPa are used. Details of the Baylis surface-wash system are shown in Figure 8-14.

A second type of surface-wash system consists of horizontal pipes located about 5 cm above the top of the filter media with plastic orifices pointing downward at an angle of 30° below the horizontal (Hudson, 1981). These units are designed for pressures of 500 KPa and apply a flow rate of 2 to 5 $m^3/m^2/hr$ to the bed surface. The orifices are spaced so that they provide complete coverage of the filter bed, as shown in the photograph of Figure 8-15. The influence of the water jets emitted from the orifices has been shown in practice to not carry laterally more than 45 cm (Hudson, personal communication).

In surface-wash systems, the piping provides a direct cross-connection between filtered and unfiltered water, if distribution system water pressure is used for the surface wash. To prevent back-siphonage, the surface-wash header should be above the filter box, and fitted with a vacuum beaker.

Design of Declining-Rate Filters

The design of declining-rate filters, including detailed examples and hydraulic calculations, is provided by

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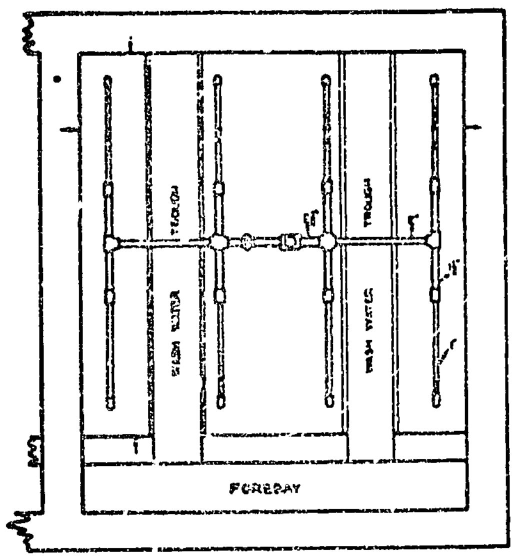
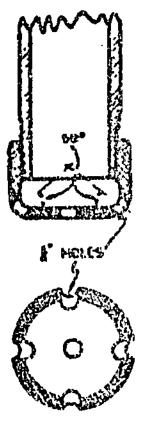
FIGURE 8-14

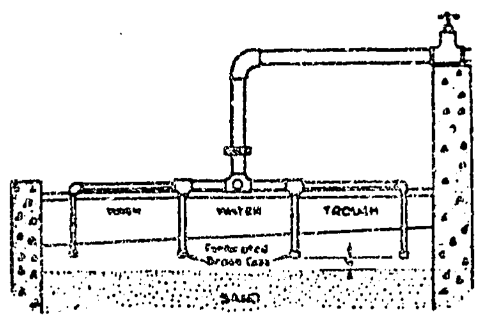
Details of Baylis Surface-wash Piping



PLAN OF SURFACE-WASH PIPING IN PLANT FILTER.



SECTIONS THROUGH PERFORATED BRASS CAP

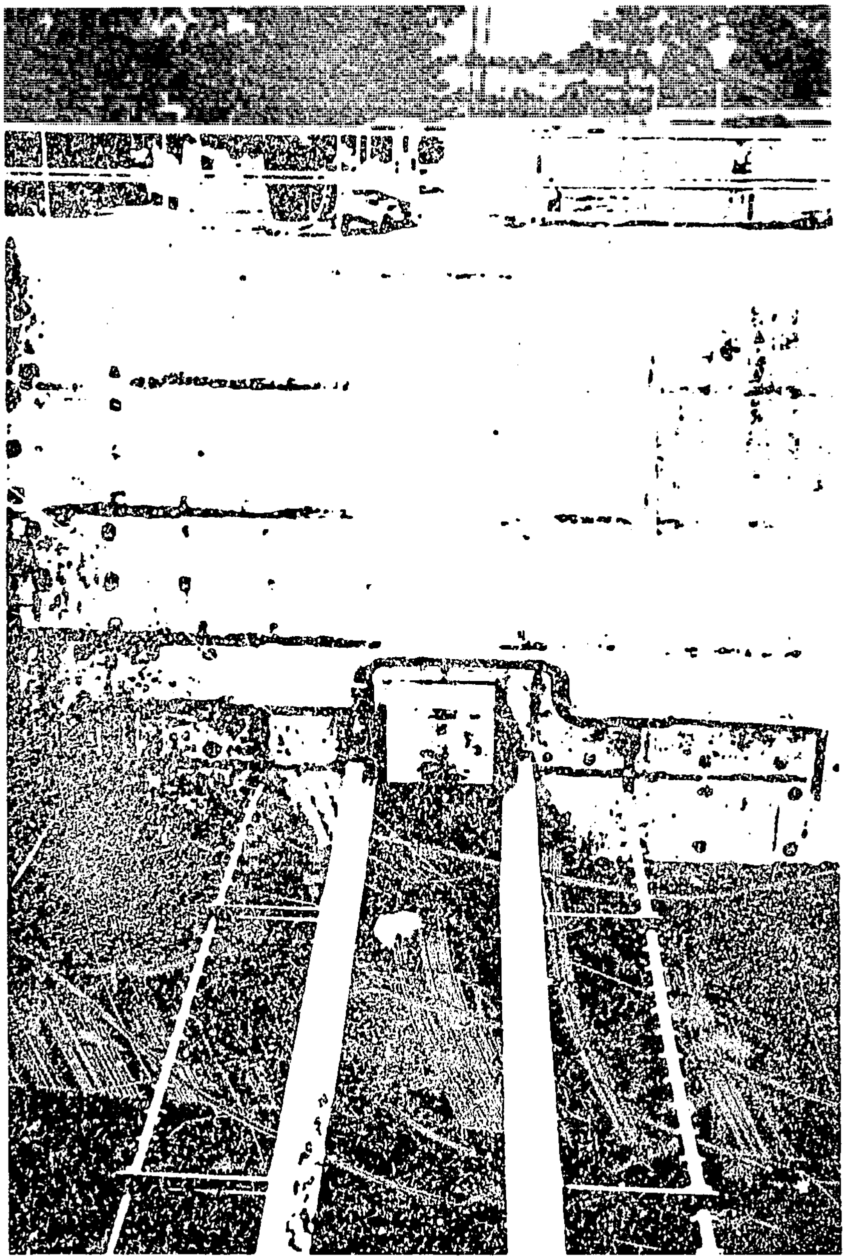


ELEVATION OF SURFACE-WASH SYSTEM PIPING IN PLANT FILTER NO. 2

[SOURCE: Hudson, personal communication]

FIGURE 8-15

Fixed-grid Surface-Wash System at a Plant in Cali, Colombia



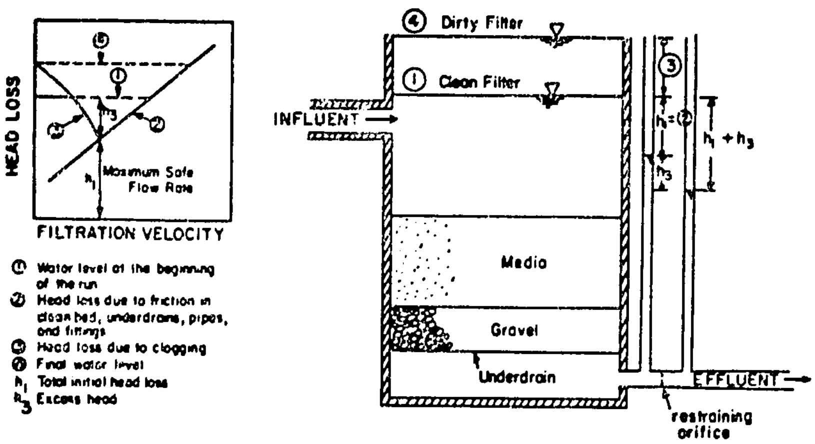
[SOURCE: Hudson, personal communication]

Hudson (1981). The design of the hydraulic control systems for declining-rate filters is covered by Arboleda (1974), who includes procedures and layouts for minimizing the use of equipment and simplifying filter operations. The discussion presented here summarizes a design procedure developed by Hudson (1981).

Declining-rate filters do not use any rate controllers, allowing the rate to decline in each unit as the head loss increases, minimizing or avoiding terminal breakthrough. The influent flow is distributed among the filters even when one filter is removed from service for backwashing. The graph of head loss vs. filtration velocity and the accompanying diagram presented in Figure 8-16 shows the basic hydraulic elements in the design of declining-rate filters. If the water level at the beginning of the filter run (line 1) is known, a vertical line can be drawn passing through the maximum safe flow rate. The initial head loss at that point (h_1) is comprised of the friction head loss in both the clean filter bed and the filter appurtenances. The excess head (h_3) is the difference between the initial head loss at that point (h_1) and the initial water level in the filter. When a filter has just been cleaned, it may be necessary to dissipate this excess head through some type of restraining orifice or adjustable gate, in order to limit filtration rates to the maximum rate of flow permitted. For example, if a rapid filter is designed for a total available

FIGURE 8-16

Heads and Water Levels
in Declining-Rate Filtration Systems



[SOURCE: adapted from Arboleda, 1974, p. 88] head ($h_1 + h_3$) of 3 meters, and the frictional head loss in the clean sand bed and appurtenances are calculated to be 2.50 meters, then the restraining orifice should be sized to produce a head loss of 0.5 meters. For a filter bed surface area of 25 square meters and an initial filtration rate of 15 m/hour, the flow rate onto the filter bed would be 9000 m^3/day; hence, from equation 8-6, the area of the orifice opening to produce a head loss of 0.5 meters would be calculated to be 0.05 m^2.

Friction head losses in the filter appurtenances have been approximated by Hudson (1981) for a rate of flow of 60 cm/min:

Filter inlet and piping	60.0 cm
Filter gravel	5.0 cm
Filter underdrains	100.0 cm
Outlet piping, filter to equalizing chamber	60.0 cm
TOTAL	2.25 meters

The Kozeny equation is used to calculate head losses in the filter media, which take place under laminar flow conditions (Fair, Geyer, & Okun, 1968):

$$h/l = (k/g)(u/p)\ v\ ([1-f]^2/f^3)\ (A/V)^2 \qquad (8\text{-}5)$$

where

h = head loss (m)

l = depth of filter bed (m)

g = gravity constant (9.81 m/sec^2)

u = dynamic viscosity (kg/msec)

p = density (kg/m^3)

f = porosity (dimensionless)

A = surface area of filter bed (m^2)

V = volume of filter bed (m^3)

v = approach velocity of the water above the sand bed (m/s)

The total available head, represented by $h_1 + h_3$ in Figure 8-16, is usually equal to the difference between the water level in the common influent header preceding the filters and the minimum water level in the filtered water outlet chamber. Hudson recommends designing for a total available head of 3 meters, unless there is assurance that filter runs will be of reasonable length under lower heads. Declining-rate filters are generally designed to operate from 150 to 50% of the average filtration rate, which ranges from 5 to 7 m/hour. However, filtration rates 2 to 3 times greater than this rate may be possible when using dual-media filters.

Design and Operation of Interfilter-Washing Units

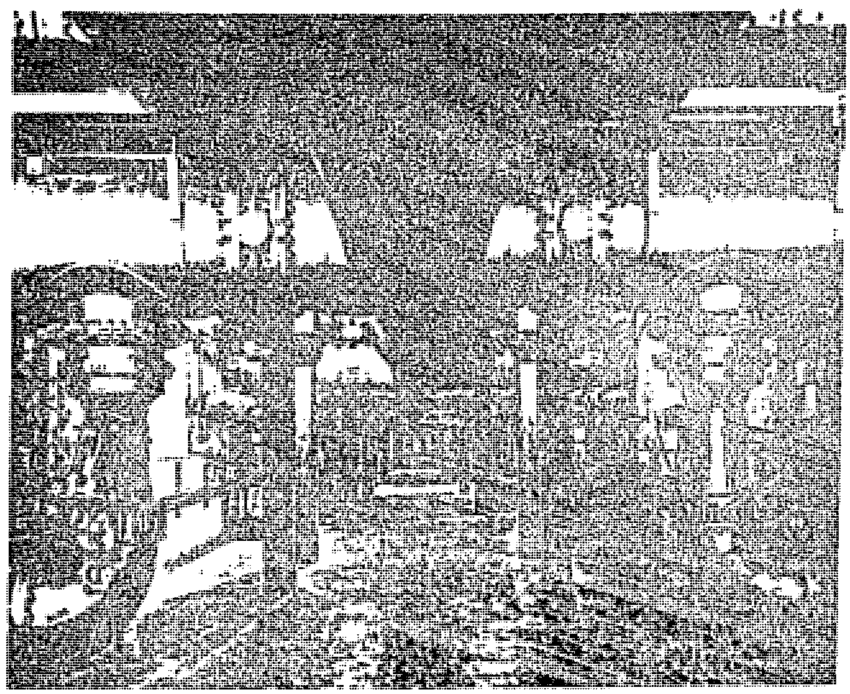
Interfilter-washing filtration units working with declining rate are an ideal type of filtration system for developing countries. Studies conducted in Latin America (Sperandio and Perez, 1976) have demonstrated the high efficiency of this type of filter, which is capable of producing greater amounts of water (more than 150%) than conventional constant-rate filters with a better effluent quality (0.50 NTU) at a lower construction cost. Moreover, such filters are easier to build, operate, and maintain than conventional filters. For example, only two butterfly valves or sluice gates are needed for filter control (three valves, if using a surface-wash system) and the entire system may be designed with concrete channels or box conduits. It is also possible to completely eliminate pipe galleries containing elaborate piping, valves, and controlling systems which are common to conventional filtration schemes. Pipe galleries like the one shown in Figure 8-17 for a conventional rapid filtration plant not only must be maintained, but also represent a major portion of the filtration complexity and cost. Other advantages of interfilter washing filtration systems are enumerated below (Arboleda, 1974):

1) Backwashing is automatically controlled by the level of the effluent weir. By changing its elevation with stop logs, it is possible to change the washwater rate.

2) The backwashing operation starts slowly with the descending level in the filter. There is no risk of an

FIGURE 8-17

Typical Filter Pipe Gallery at a
Conventional Filtration Plant in the US



[SOURCE: UNC Health Sciences Library, personal communication] abrupt start in the expansion of the filter bed which might disturb the media.

3) If the filters are not backwashed at the proper time, the plant flow decreases and there is a "backwater" effect, which forces the operator to act immediately.

4) There is no possibility of producing a negative head loss.

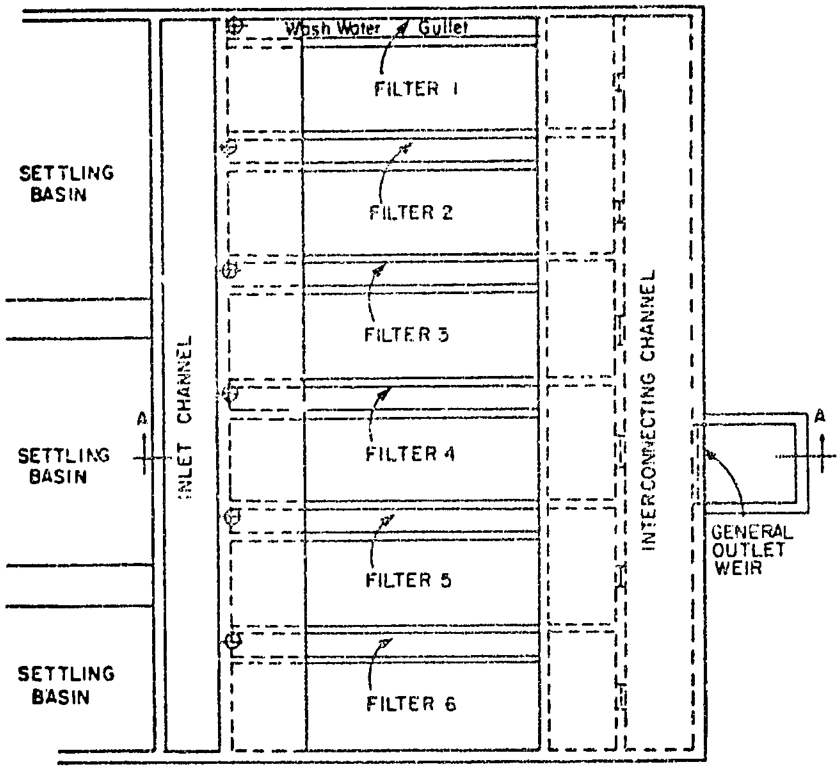
5) The underdrain system can be inspected.

Interfilter-washing units have been planned or are operating successfully in large plants in Latin America, including those for the cities of Monterey, Mexico (1.04 million m^3/day); Mexico City (1.08 million m^3/day); Rio Grande, Brazil (518,000 m^3/day); Santo Domingo, Dominican Republic (691,000 m^3/day); and Cali, Colombia (259,000 m^3/day); as well as in at least 100 smaller plants. The hydraulic characteristics of these filters have been described by Arboleda (1972, 1973), who has also developed several innovative and simple filter layouts. The material presented is based largely on his published works.

Interfilter-washing units are designed with either unrestricted or restricted declining flow rate. The filtration system for the plant in Cochabamba, Bolivia, a plan of which is shown in Figure 8-18, employs unrestricted declining flow rate. A section of a typical filter cell in the plant, which shows the water levels during filtration, is shown in Figure 8-19. At the start of the filter cycle,

FIGURE 8-18

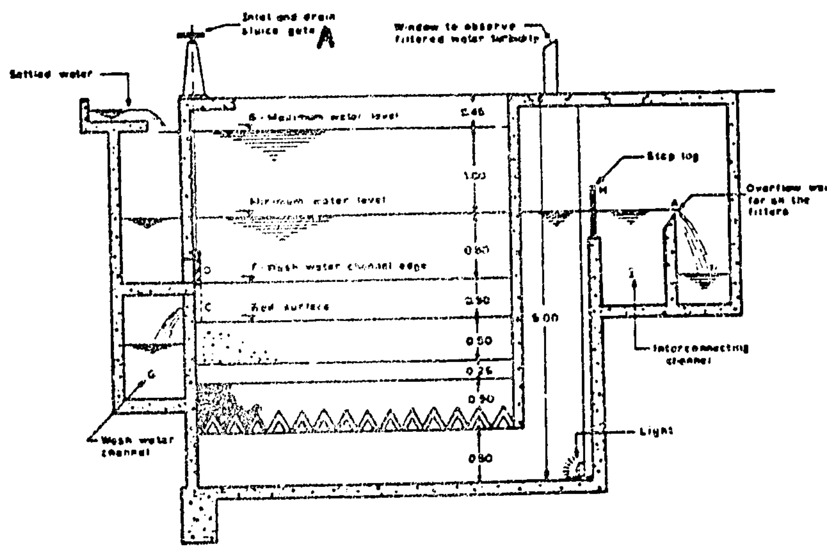
Battery of Interfilter Washing Cells
at the Plant in Cochabamba, Bolivia



[SOURCE: Arboleda, 1974, p. 90]

FIGURE 8-19

Typical Filter Cell at the Cochabamba Plant,
Showing Water Levels During Filtration



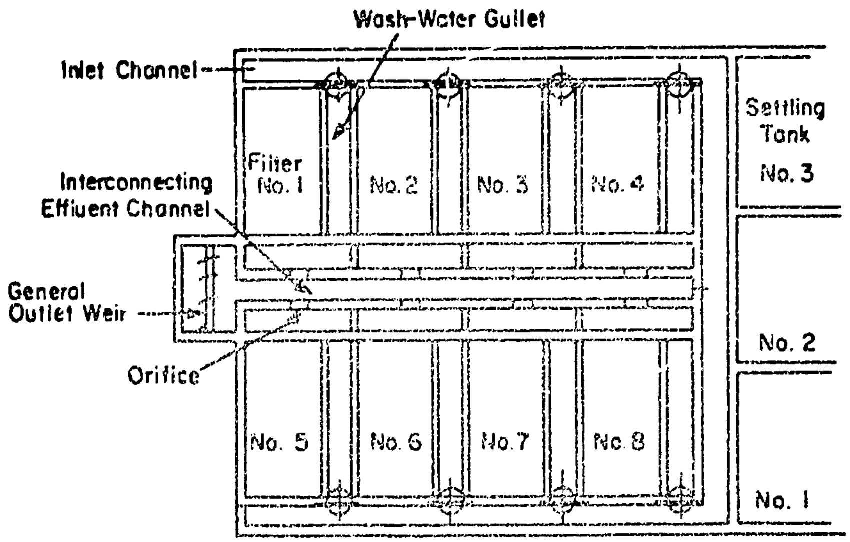
[SOURCE: Arboleda, 1974, p. 90]

the minimum water level is established slightly above the surface of the water flowing over the general outlet weir, in accordance with the initial head loss. The water slowly rises in the filter box as head loss builds up during filtration, until the maximum allowable head loss is reached (normally 1.5 to 2.5 meters), at which point the filter must be backwashed. To initiate the backwash cycle, only one sluice gate need be manipulated. In Figure 8-19, gate A slides upwards opening the drain C and closing the inlet D. The rate of backwashing is controlled by adjusting the level of the general outlet weir. In order that newly cleaned filters do not take an excessive load when put back into operation, all filter units should always be kept reasonably clean so that the filtration rate in any filter does not exceed 30 m/hr at the beginning of the run. This is best achieved by washing the filters in succession on a time schedule.

The second type of design introduces a constriction at the outlet side of the filter by means of a sluice gate that can be adjusted to control the filter velocity after washing. A plan view of a battery of filters that employ restricted declining flow rate is shown in Figure 8-20; section views that show the water levels during filtration and backwashing are shown in Figures 8-21 and 8-22, respectively. During the filtration cycle (Figure 8-21), the head losses due to the filter bed and underdrain system,

FIGURE 8-20

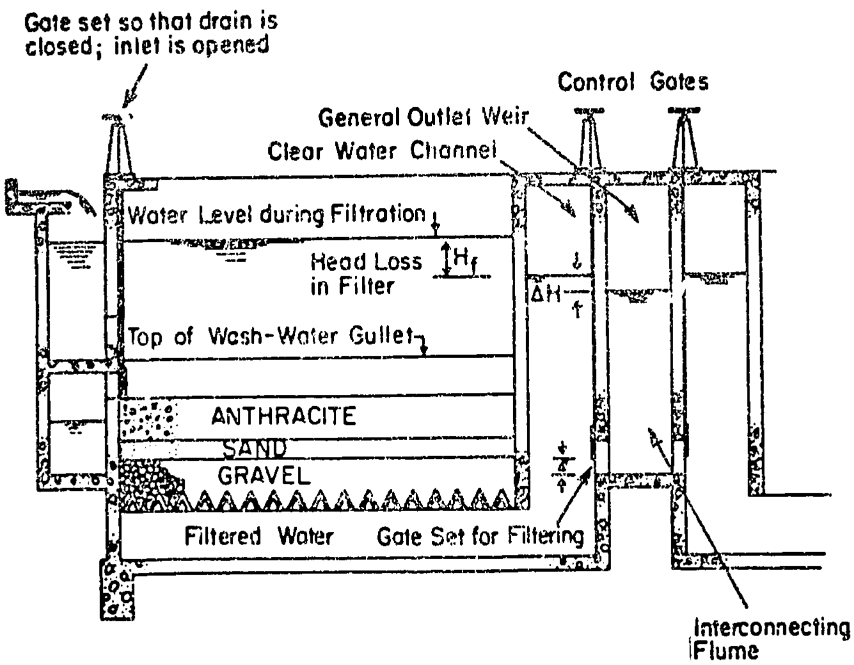
Battery of Interfilter Washing Cells
with Outlet-Orifice Control (plan)



[SOURCE: Arboleda, 1973, p. 446]

FIGURE 8-21

Typical Filter Cell with Outlet-orifice Control,
Showing Water Levels During Filtration (cross-section)



[SOURCE: Arboleda, 1973, p. 446]

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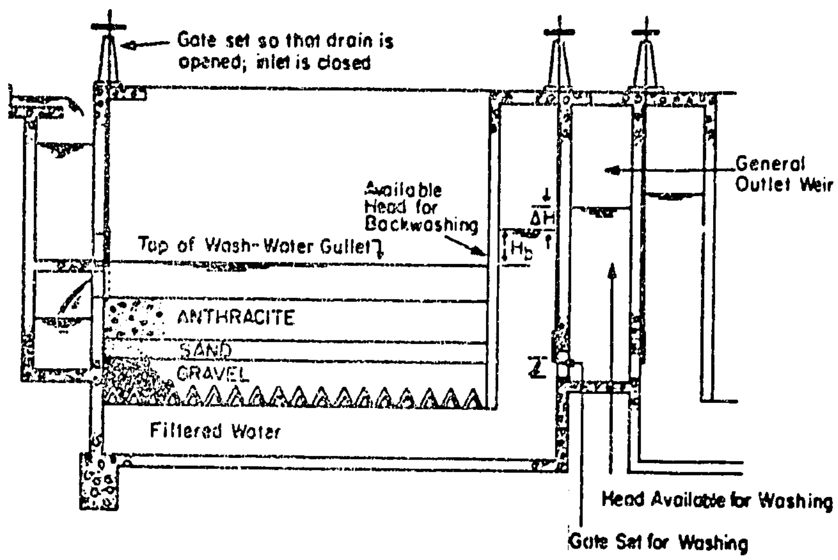
FIGURE 8-22

Typical Filter Cell with Outlet-orifice Control, Showing Water Levels During Backwashing (cross-section)



[SOURCE: Arboleda, 1973, p. 449]

H_f, dictate the water level in the clear-water channel which under normal operation is the same for all filter units. The difference in water level, ΔH, between the clearwater channel and the interconnecting flume is controlled by the outlet sluice gate, and dictates the rate of filtration. During the backwash cycle (Figure 8-22), the same gate is used to control the head available for backwashing, H_b, and thus the rate of rise of washwater. To change from a filtration to backwash mode, and vice versa, only two gates are used: (1) the gate located at the inlet side of the filter which controls the opening and closing of the inlet channel and drain; and (2) the gate located at the outlet side of the filter which regulates the filtration and washwater rates. The water level difference, ΔH, controlled by the outlet sluice gate can be recorded to measure flowrates through the constriction. From the position of the gate, the opening between the two chambers would be known, and the flow an be calculated from the following equation:

$$Q = CA\ (2g\,\Delta H)^{1/2} \qquad (8\text{-}6)$$

where

Q = flowrate (m^3/sec)

C = orifice coefficient (typical values 0.7-0.8)

g = gravity constant (9.81 m/sec^2)

ΔH = water level difference between chambers (m)

A = cross-section area of orifice (m^2)

Hudson (1981) has designed a declining-rate interfilter-washing filtration system whereby the flow is restricted on the inlet side and butterfly valves are used in place of sluice gates. This design was used for the treatment plant in Cali, Colombia. A schematic of a typical filter cell in the plant is shown in Figure 8-23. The filters receive water from a relatively deep inlet channel. From this channel the water flows through influent pipes, each with an open-close butterfly valve and restraining orifice. Each cell also has a drain valve. The filtered water discharges over a common effluent weir. Hudson claims the following advantages for this type of system:

(1) The inlet constriction allows a changing flow rate to be applied to the system, and also provides a declining flow rate through each filter as the filtration progresses;

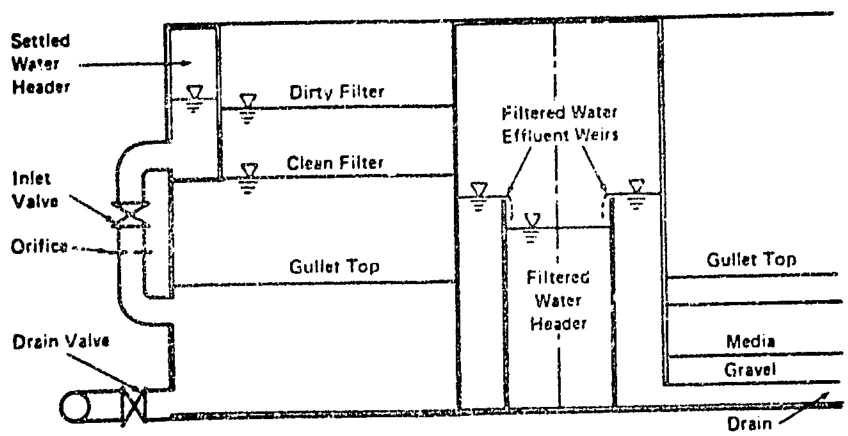
(2) Valve structure is required only at the inlet end to regulate the flow supply or the backwashing operation;

(3) Butterfly valves are simpler and faster to operate than sluice gates and are easily maintained, since all parts are exposed and accessible. Butterfly valves are also generally less expensive.

Several design guidelines that should be taken into account when designing interfilter-washing filtration systems are summarized below:

FIGURE 8-23

Influent-controlled, Declining-rate Filter System
for the Plant in Cali, Colombia (cross-section)



[SOURCE: Hudson, 1981, p. 195]

1) The capacity of and the flow through the plant must be at least equal to the washwater flow needed to clean one filter.

2) A minimum of four filters, each capable of operating at one-third higher rate, is necessary to operate at design capacity when one unit is out of service for washing.

3) The filters must be so designed that any one may be taken out of service for repairs without interruption of the normal operation of the others.

4) The underdrain system must be especially designed to produce low head loss. This is feasible because the filters are completely open at the bottom, and the washwater flow velocity is therefore very low.

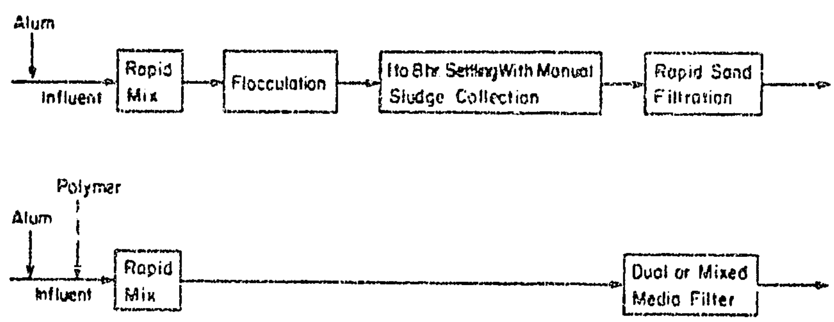
5) The influent channel should be able to carry the flow to any filter unit, at any time required, with a low loss of head.

Direct Filtration

Direct filtration of raw waters low in turbidity and color is a comparatively low-cost option that has distinct advantages for developing countries. The direct filtration process subjects raw water to rapid mixing of coagulants, and sometimes flocculation, followed directly by filtration. Figure 8-24 shows separate flow sheets for a conventional filtration plant and a direct filtration plant, the latter

FIGURE 8-24

Flow Sheets Comparing Conventional Filtration (using alum) with Direct Filtration (using alum and a nonionic polymer)



[SOURCE: adapted from Culp, 1977, p. 375]

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consisting of the addition of alum and a polymer to the rapid mix influent, followed by dual-media filters.

The chief advantage of direct filtration for developing countries lies in the potential for reduced chemical consumption and resulting reduced sludge load. Reduced chemical costs and reduced sludge handling can significantly lower the operation and maintenance costs of the plant, especially if chemicals must be imported from abroad. Plants designed with direct filtration also have lower capital construction costs than conventional plants; up to 30% under certain conditions (Culp, 1977), which results from the elimination of settling basin structures, sludge removal equipment, and flocculation structures and equipment. Other advantages include lower operation and maintenance costs due to the elimination of this equipment, and the simplification of collecting waste solids as they are contained in the filter-backwash water.

Conventional filtration plants with horizontal-flow settling basins may also be operated in the direct-filtration mode by reducing the basic coagulant dosage in the rapid mix, thereby reducing or preventing sedimentation in the settling basin. Of course, the plant always has the option of reverting back to the conventional mode if, for example, seasonal flooding raises the raw water turbidity above acceptable levels for direct filtration.

The application of direct filtration is generally limited to raw waters having the following characteristics:

1) The raw water turbidity and color are each less than 25 units;

2) The color is low and the maximum turbidity does not exceed 200 NTU; or

3) The turbidity is low and the maximum color does not exceed 100 units.

Pilot plant studies should be performed in each case to determine if the water can be treated successfully by direct filtration.

Wagner and Hudson (1982) have developed a simple bench-scale test that evaluates the possibility of using direct filtration. Basically, the method utilizes jar testing to sort out the variables of best coagulant, most effective polymer, optimum dosages, and stirring intensities and time. Samples are taken from each of the jars, and then filtered through standard laboratory filter paper. The filter test run takes no more than 2 or 3 minutes. When the coagulant dose required to produce a low filtered water turbidity is less than 6 to 7 mg/l with the addition of a small dose of polymer, the raw water has the potential to be treated by direct filtration. When higher doses are required, say 15 mg/l, then treatment by direct filtration is doubtful. Positive results obtained from bench-scale testing justify undertaking more conclusive pilot-filter testing to determine the plant-scale filter design parameters.

The design of direct filtration plants is concerned primarily with the design of the rapid-mix units and the filters. Ordinarily, flocculation is not necessary if a properly designed rapid mixer is provided and filters with relatively fine media and deep beds are used. Dual filters are generally used, with filtration rate of about 1.1 m/hr, although rates as high as 2.5 to 18 m/hr have been used in practice. Dual-media filters should be composed of the finest filter medium that can be used without unduly short filter runs (less than 8 hours), as determined by pilot filter tests. A typical 90 cm deep dual-media filter for direct filtration consists of 62 cm of anthracite, effective size of 0.8 mm and specific gravity of 1.55; and 28 cm of sand, effective size of 0.45 mm and specific gravity of 2.4 (Culp, 1977). Filter runs are generally shorter than those encountered in conventional filtration plants. Also, washwater usage may be as high as 6% of plant output as compared to about 1 to 4% for conventional filters.

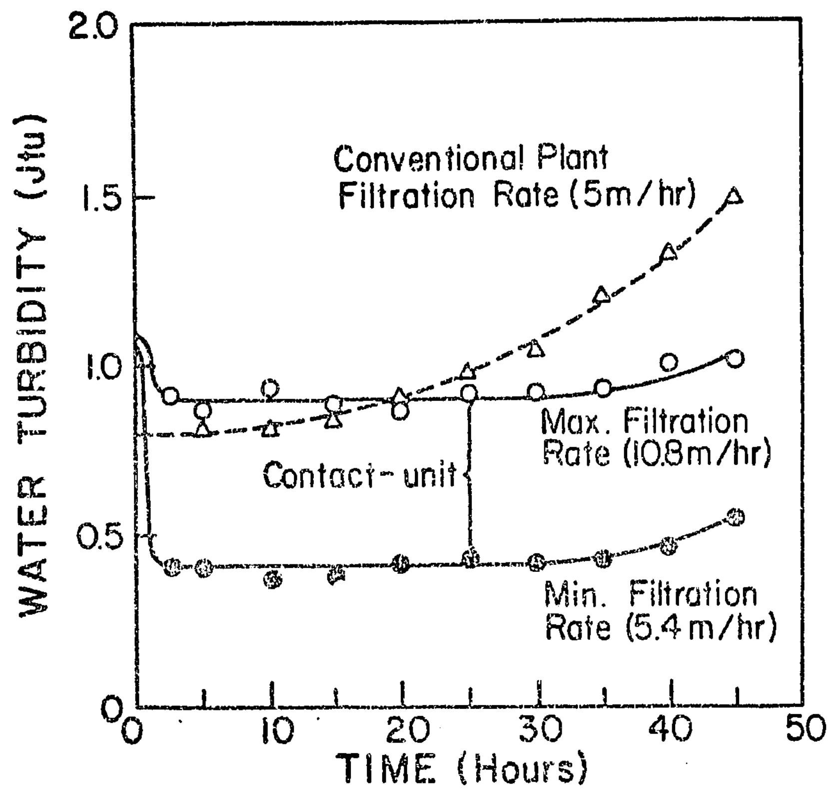
A comparative study of the efficiencies of direct and conventional filtration was conducted in a plant serving the city of Linhares, Brazil (Sperandio and Perez, 1976). The plant contained a rapid mixer followed by two independent treatment processes operated in parallel: conventional rapid filtration working with declining rate and direct filtration using upflow filters. The raw water turbidity entering the plant averaged 11 NTU and the color averaged 23 color units. The upflow filter contained a sand bed 2 meters deep having an effective size of 0.6 mm and uniformity coefficient of 2.5. In general, the direct-filtration upflow unit (or contact unit) was more efficient, as shown in the comparative graph in Figure 8-25. Two filtration rates were used in the upflow unit, a minimum rate of 5.4 m/hr and a maximum rate of 10.8 m/hr. The filter runs averaged about 30 hours. The filters were backwashed at an upflow velocity of 36 m/hr for a period of 20 minutes, owing to the great depth of the sand bed (twice the normal depth). Although no figures were given in this report, the coagulant doses were reported to be much lower than those for the conventional filters.

Upflow-Downflow Filtration

Upflow-downflow filtration is a simple and economic treatment method for smaller water treatment plants in developing countries. In this type of system, a battery of upflow roughing filters (called contact clarifiers) replaces the conventional arrangements for mixing, flocculation, and sedimentation used in rapid filtration plants. This can result in reduced construction and operation costs; the latter because the coagulant dosage used with this type of

FIGURE 8-25

Comparative Efficiencies of the Conventional Plant and Contact Unit at the Plant in Linhares, Brazil



[SOURCE: Sperandio and Perez, 1976]

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design is generally smaller than that used for conventional treatment. Upflow-downflow type filters may be designed to treat turbid waters as well as relatively clean waters with the typical designs described in this section. It should be noted, however, that in places where there are sudden variations in the raw water quality, the amount of time available for adjusting the coagulant dosage is smaller than that for conventional treatment since, by comparison, the water being treated flows quickly through the plant. An important application of upflow-downflow filtration is in the design of simple modular and package water treatment plants. All of the designs described herein can be so adapted for fabrication of modular plants, which will further reduce their cost, and enable prefabricated units to be transported to remote areas where on-site construction is impracticable (see Chapter 9).

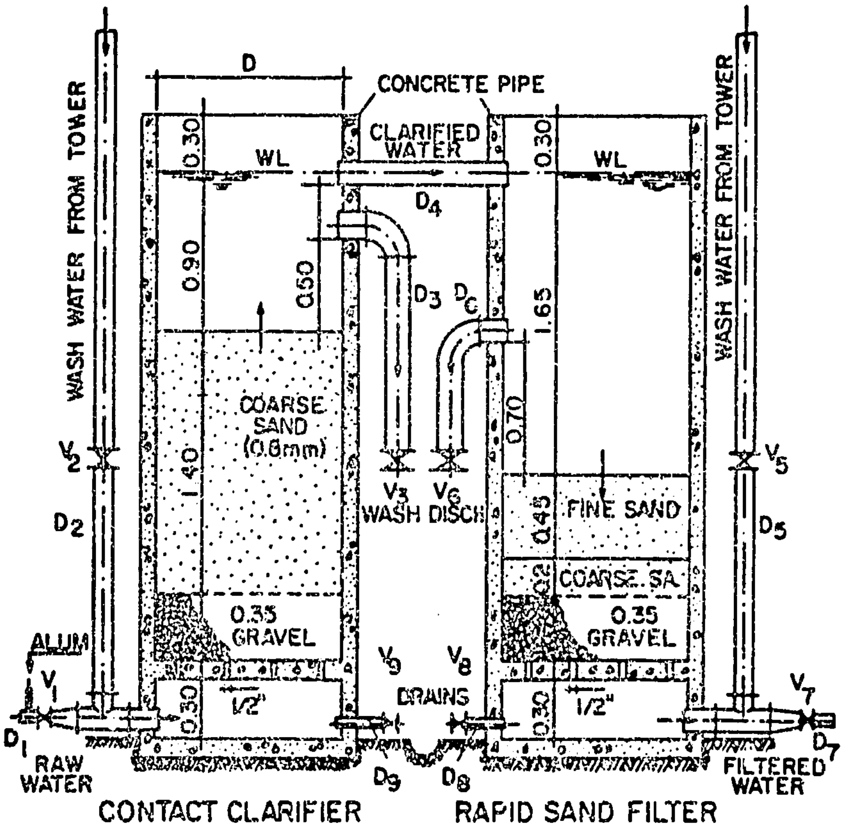
The filter medium of the upflow contact clarifier may range from coarse sand having an effective size of 0.7 to 2.0 mm, up to graded gravel ranging in size from about 10 mm to 60 mm. The depth of the contact bed should be between 1.5 and 3 meters. For coarse sand beds, filtration rates as high as 12 to 16 m/hr may be used whereas, to prevent excessive floc carryover to the filters, those for gravel beds are limited to 4 to 8 m/hr. The choice of media for the contact clarifier should be based on pilot-filter studies of the water to be treated. The design parameters for the downflow (or polishing) filter are analogous to those used for rapid filters. Dual-media filter beds are preferred to allow for higher filtration rates and longer filter runs. Backwash arrangements are necessary for both the contact clarifier and polishing filter. Water pressure for backwashing may be provided by an elevated washwater tank.

A simple upflow-downflow filtration scheme involving only two concrete pipes is shown in Figure 8-26 (Azevedo-Netto, 1977). Table 8-7 indicates the required structural dimensions and washwater flow rates for this double filtration unit, called "superfiltration," for capacities ranging from 100 to 450 m^3/day. For larger installations, the capacity may be increased by placing a battery of superfilters in parallel. For efficient treatment with superfiltration, raw water turbidity should not exceed 160 NTU and normally be less than 50 NTU, while the color should not exceed 80 color units.

Superfilters were installed recently in the city of Colon, Costa Rica and two smaller communities in that country (Institute of Water Supply and Sewerage, Costa Rica; personal communication). Several faults in the design and construction of these units have led to numerous operational problems, the most important being inefficient distribution of washwater and excessive backwash rates that have resulted in loss of anthracite filter media, and clogging of the inlet pipes to both filters. Furthermore, the overall

FIGURE 8-26

Gravity "Superfilter" - Brazil



[SOURCE: Azevedo-Netto, personal communication]

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TABLE 8-7: Design Guidelines for Upflow-Downflow Filtration Units in Brazil[a]

FLOW (m3/day)	$D_1 = D_7$ (cm)	$D_2 = D_5$ (cm)	$D_3 = D_6$ (cm)	D_4 (cm)	$D_8 = D_9$ (cm)	DIAMETER D (cm)	WASHING (liter/sec)
100	5	10	15	15	4	100	12
150	5	10	15	20	4	120	17
250	7.5	10	15	25	4	150	30
450	7.5	15	20	30	5	200	50

[a]For the significance of the various symbols, see Figure 8-25

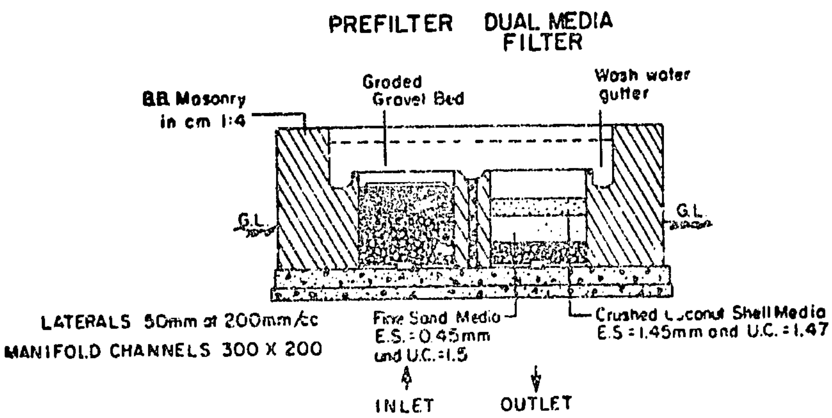
[SOURCE: adapted from Azevedo-Netto, 1981, personal communication] performance of the superfilters was generally poor, with about 50 to 90% turbidity removal when treating raw water with a turbidity less than 50 NTU. Filter runs averaged typically about 10.5 to 12 hrs. The difficulties described above were attributed primarily to improper hydraulic design of the filters and ancillary piping.

Plans and specifications for two types of upflow-downflow treatment plants have been published recently by CEPIS (1982), for capacities ranging from 864 to 1680 m^3/day. In both design, raw water and backwash waters are delivered to a battery of three upflow filters form an elevated storage tank via either a single or dual-pipe system. In the single-pipe system, when an upflow filter has to be washed, operation of the remaining units must be interrupted by closing appropriate valves in order to provide sufficient flow of washwater to the dirty filter. In the dual-pipe system, the storage tank has two compartments: one for water to be filtered and one for backwash water. Each tank is provided with a pipe system connected to the filters, hence the flow of raw water to the upflow filters need not be interrupted while one of the filters is being backwashed. The battery of four downflow filters which follow the upflow filters are characterized by declining-rate filtration, dual-media beds, and interfilter-washing capabilities. Wherever possible, concrete distribution channels have been used in place of pipes in the plant. Raw water quality restrictions are the same as those for super-filtration. Plants similar to the designs developed by CEPIS have been installed in several Brazilian states.

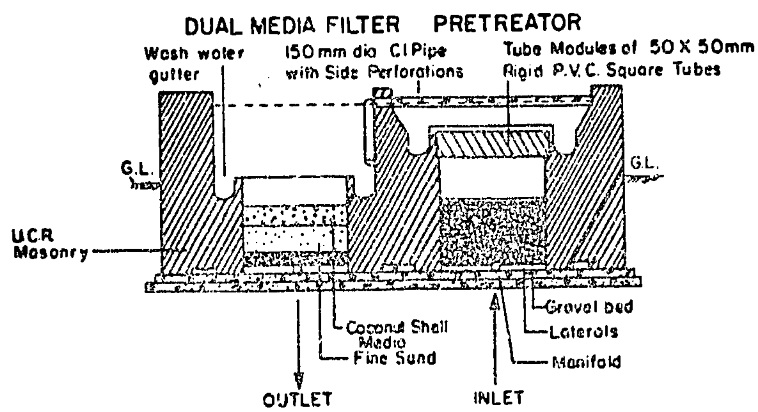
Kardile (1981) has developed three upflow-downflow filtration designs for rural communities in India. The plant for the community of Ramtek (population of 20,000) was designed for the treatment of low turbidity waters, whereas the plants for the communities of Chandori (population of 15,000) and Varangaon (population of 35,000) were designed for the treatment of turbid waters. The Ramtek plant has a capacity of 2400 m^3/day and is comprised of a gravel bed-cum-tube settler pretreatment unit followed by a dual-media filter. The Chandori plant has a capacity of 1000 m^3/day and is comprised of a gravel-cum-tube settler pretreatment unit followed by a dual-media filter. The Varangaon plant has a capacity of 4200 m^3/day and is comprised of two treatment units in parallel consisting of one gravel bed flocculator and tube-settling tank, which are followed by three dual-media filters. Flow diagrams for the Ramtek and Chandori plants are shown in Figure 8-27; the Varangaon plant is shown in Figure 8-28. Recommended design criteria for each of these plants are listed in Table 8-8. The design of gravel-bed flocculators, which are common to all three plants, is discussed in Chapter 6 ("Gravel-Bed Flocculators"). The dual-media filters for each of the

FIGURE 8-27

Upflow-Downflow Filtration Plants in India
a) Flow Diagram of Ramtek Filter; b) Flow Diagram of Chandori Plant



a. Flow diagram of Ramtek filter.



b. Flow diagram of Chandori treatment plant.

[SOURCE: Kardile, 1981, p. 226]

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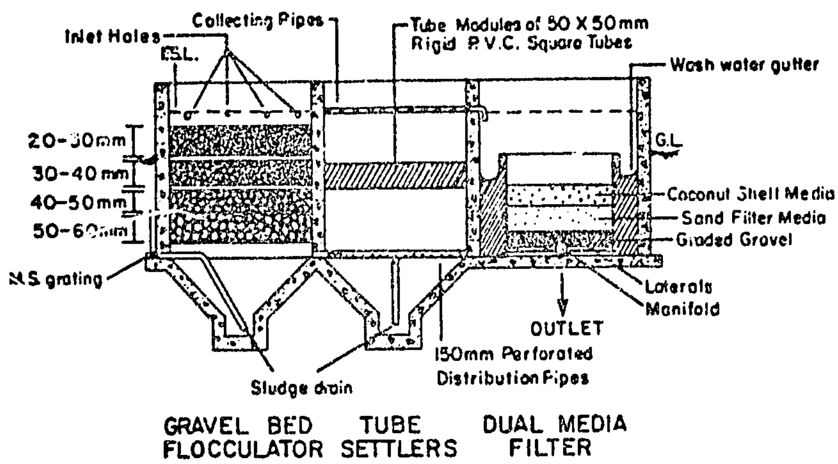
FIGURE 8-28

Flow Diagram of Upflow-downflow Plant in Varangaon, India



[SOURCE: Kardile, 1981, p. 227]

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TABLE 8-8: Recommended Design Criteria for the Indian Upflow-Downflow Treatment Plants

	RAMTEK PLANT DESIGN CRITERIA	VARANGAON PLANT (2400 m^3/day)	CHANDORI PLANT (4200 m^3/day)	(1000 m^3/day)
		I. RAW WATER TURBIDITY		
i)	General Recommendations	For low turbidity sources	For high turbidity sources	For moderate turbidity sources
ii)	Average range in NTU	10 to 30	30 to 100	30 to 100
iii)	Maximum range in NTU	300 to 500	1000 to 5000	1000 to 2000
		II. PRETREATMENT		
A)	Mixing unit	Mixing channel	Mixing channel	Mixing channel
i)	Type of gravel bed units	Prefilter	Flocculator	Pretreator
ii)	Direction of flow	Upward	Downward	Upward
iii)	Surface loading in m/hr	4 - 7	4 - 1	4 - 8
iv)	Depth of the gravel bed in m	1.5 to 2.0	2.5 to 3.0	1.5 to 2.0
B)	Tube settling tank	Not adopted	Tube settler	Gravel bed-cum-tube settler
i)	Surface loading in m/hr	--	5 to 10	4 to 8
ii)	Detention period in minutes	--	30 to 50	30 to 50
iii)	Depth of the tank in m	--	3 m above hopper	3.5 to 4.0
iv)	Direction of Flow	--	Upward	Upward
v)	Size of PVC square tubes	--	50 mm x 50 mm	50 mm x 50 mm
vi)	Depth of tube settler	--	0.5 to 0.6 m	0.5 to 0.6 m
		III. DUAL MEDIA FILTER BED		
i)	Surface loading in m/hr	4 to 7	5 to 10	4 to 8
ii)	Dual media details			
	a) Coconut shell media depth	30 to 40 cm	30 to 50 cm	30 to 50 cm
	average size in mm	1.0 to 2.0	1.0 to 2.0	1.0 to 2.0
	b) Fine sand media depth	50 to 60 cm	50 to 60 cm	50 to 60 cm
	Effective size in mm	0.45 to 0.55	0.45 to 0.55	0.45 to 0.55
	Uniformity coefficient	Below 1.5	Below 1.5	Below 1.5
iii)	Back wash method	Hard wash	Hard wash	Hard wash

[SOURCE: adapted from Kardile, 1981, p. 229] plants consist of a layer of crushed coconut shells (specific gravity 1.4) placed on top of layers of sand and gravel. The underdrain system is a simple perforated pipe and manifold design that is manufactured locally. The thick masonry sidewalls, clearly outlined in Figure 8-20 for the Ramtek and Chandori plants, were used in place of reinforced concrete to take advantage of local materials and unskilled labor found in the villages. They also served to support the rapid-mixing channels and walkways. The construction costs for the three plants were between 30 and 50% of the construction costs for the same capacity conventional rapid filtration plants (see Chapter 10, "Construction Costs of Water Treatment Plants").

An evaluation of the performance of the Varangaon treatment plant was conducted over a three-year period, 1978-81. Performance criteria included (1) reductions in turbidity; (2) washwater consumption; and (3) length of filter run (Tasgaonkar, 1982). Turbidity readings for raw, settled, and filtered waters, together with corresponding alum doses, are presented in Table 8-9 for several randomly selected dates. Despite some periods of extremely high raw water turbidities (e.g. 10,200 NTU was recorded during a monsoon on March 29, 1979), the plant was still able to produce filtered water with turbidities below the current Indian standard of 2.5 NTU. Alum consumption varied directly with raw water turbidity, and in some instances was quite high (208 mg/l on March 29, 1979). However, the average alum consumption over a period of a year did not indicate that the simplified plant consumed more alum than conventional plants. The washwater consumption as a percentage of total filtered water varied from 1.4 to 2.9%. The average length of filter runs was 45 hours.

TABLE 8-9

Turbidity of Raw, Settled, and Filtered Water for the Varangaon Plant - India

Date	Raw	Settled	Filtered	ALUM DOSE (mg/l)
	TURBIDITY OF WATER (NTU)			
August 1978	5,100	22	1.8	144
August 1978	9,610	75	2.1	196
September 1978	2,810	20	1.8	112
October 1978	28	13	1.5	10.6
March 1979	30	17	1.6	8.0
August 1979	10,200	29	2.2	208
August 1979	4,700	24	2.0	120

Slow-Sand Filtration

The flow rates for slow-sand filters are about 20 to 50 times slower than for rapid filters. Because the filter is cleaned by manually, removing the dirty top sand rather than backwashing, the sand is not stratified and its hydraulic characteristics are governed by the finer portions of the

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sand. Another distinguishing feature of slow-sand filters is the presence of a thin layer, called the "schmutzdecke," which forms on the surface of the sand bed and includes a large variety of biologically-active microorganisms. These break down organic matter, and also fill the interstices of the sand so that solid matter is retained quite effectively. The impurities present in the raw water are removed almost entirely in the upper 0.5 to 2 cm of the filter bed. The cleaning of the filter bed is carried out by scraping off this top layer when it becomes too clogged with impurities. Unless the water being treated is excessively turbid or has high algal concentrations, slow sand filters may run continuously for a period of several months before cleaning is necessary. The filter-cleaning operation may be carried out by unskilled laborers using hand tools, and completed in 1 or 2 days. Figure 8-29 shows the manual cleaning of a slow sand filter in India. After cleaning, about 1 or 2 days are further required to ripen the "schmutzdecke," and return the filter effluent quality to its former level.

The principal use of slow sand filtration is in the removal of organic matter and pathogenic organisms from raw waters of relatively low turbidity. The biological treatment that takes place in the "schmutzedecke" of the filter is capable of reducing the total bacteria count by a factor of 10^3 to 10^4 and the _E. coli_ count by a factor of 10^2 to 10^3(IRC, 1981b). Accordingly, considerable savings

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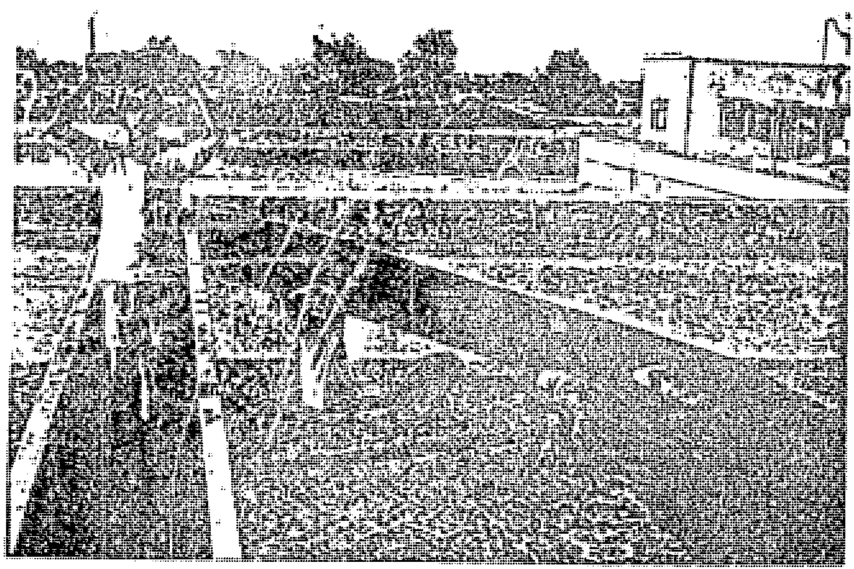
FIGURE 8-29

Manually Cleaned Slow Sand Filter in India



[SOURCE: IRC, 1978, p. 12]

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can be realized in the quantities of chlorine required for disinfection. Such an advantage is particularly important in rural areas of developing countries where chlorination practices have proven to be very unreliable, and where slow-sand filtration can provide a more reliable safety barrier than, for example, rapid filters that require uninterrupted chlorination to assure safety.

Slow-sand filters are most practical in the treatment of water with turbidity below 50 NTU, although much higher turbidities (100 to 200 NTU) can be tolerated for a few days. The best purification occurs when the turbidity is below 10 NTU (Huisman and Wood, 1974). When higher turbidities are expected, slow-sand filters should be preceded by some type of pretreatment (see Chapter 3). Although these units are thought to be outmoded, London continues to build such plants, with roughing filters for pretreatment.

Slow-sand filters provide a number of distinct advantages for developing countries which are summarized here (Feachem, McGarry, and Mara, 1977):

1) The cost of construction is low, especially where manual labor is used.

2) Simplicity of design and operation means that filters can be built and used with limited technical supervision. Little special pipework, equipment, or instrumentation is needed.

3) The labor required for maintenance can be unskilled as the major job is cleaning the beds, which can be done by hand.

4) Imports of material and equipment can be negligible and no chemicals are required.

5) Power is not required if gravity head is available on-site, and there are no moving parts or requirements for compressed air or high-pressure water.

6) Variations in raw water quality and temperature can be accommodated provided turbidity does not become excessive; overloading for short periods does no harm.

7) Water is saved - an important matter in many areas - because large quantities of washwater are not required.

The factors that weigh against the use of slow-sand filtration and lead to the choice of rapid filters as the more appropriate treatment method apply mainly in the industrialized countries; namely (1) the large land requirements (about 5 times that required for rapid filtration plants); (2) the higher construction costs in countries where construction methods are largely mechanized and labor is expensive; (3) the higher costs for cleaning the filters in countries where manual labor is expensive; (4) the need to cover the filters in freezing climates (it also is difficult to find men who will work at cleaning in cold weather); (5) the working of the biological layer, i.e. "schmutzedecke," may be upset by certain types of toxic industrial wastes or heavy concentration of colloids; and (6) certain types of algae may interfere with the working of the filters, usually choking the filter bed, which calls for frequent cleaning. This problem may be ameliorated by covering the filter or using an algicide to inhibit algal growths. Interestingly, these limitations (with the exception of the last) do not generally apply in developing countries. Of course, the large quantity of media required can raise problems in areas where suitable sand is not locally available. Under suitable circumstances, then, slow-sand filtration is the cheapest, simplest, and most efficient method of water treatment for many types of surface waters in developing countries.

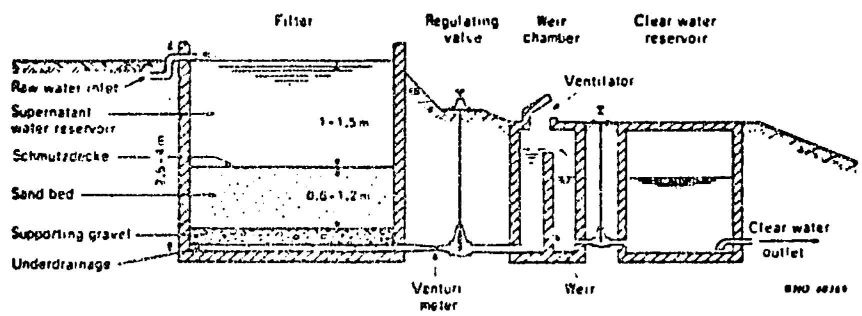
Design of Slow-Sand Filters

The essential parts of a slow-sand filter are shown in Figure 8-30. Slow-sand filters, because they are not backwashed, are much simpler in design than rapid filters. Pertinent design criteria for the design of slow-sand filters are summarized below (Huisman and Wood, 1974):

1) Rate of filtration -- The traditional rate of filtration used for normal operation is 0.1 m/hour, although it is possible to produce safe water at rates as high as 0.4 m/hour (Huisman and Wood, 1974). At higher filtration rates, the intervals between filter cleanings are shortened, but the quality of the treated water does not deteriorate. Higher rates of filtration are used during those periods

FIGURE 8-30

Diagram of a Slow Sand Filter



[SOURCE: Huisman and Wood, 1974, p. 18]

when some filters are out of service for cleaning, rather than providing extra filter units at increased costs to maintain a lower rate.

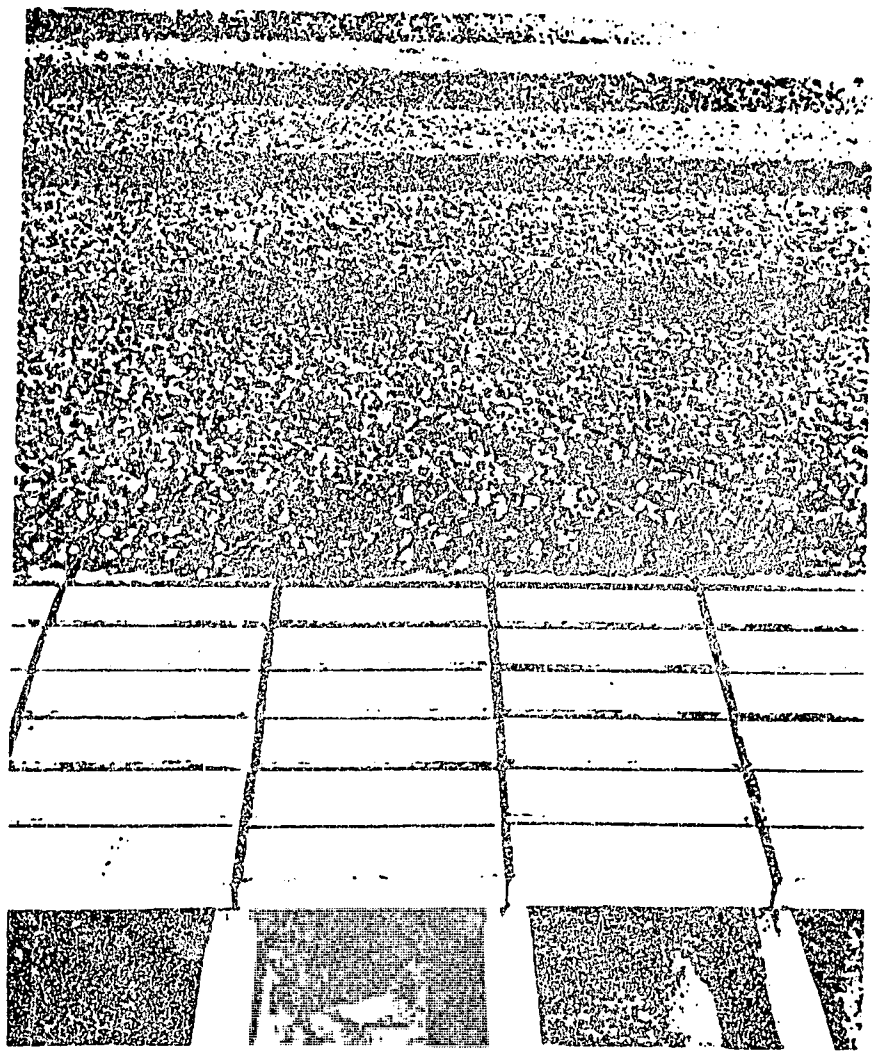
2) <u>Supernatant water layer</u> -- The depth of water should provide a head sufficient to overcome the resistance of the filter bed and prevent air binding. In practice a head of between 1.0 to 1.5 meters is usually selected.

3) <u>Filter bed</u> -- The sand bed thickness varies between 1.0 to 1.4 meters. This thickness should be reduced to not less than 0.5 to 0.8 meters after removing the upper sand layers during filter cleaning. Filter sand should have an effective size between 0.15 to 0.35 mm and a uniformity coefficient between 1.5 and 3, although a coefficient of less than 2 is desirable. The careful selection and grading of sand is not as critical as in rapid filters. Use of builder grade or locally available sand can reduce costs.

4) <u>Filter gravel</u> -- The filter gravel should be so graded that the sand does not penetrate the underdrain system, yet provides free flow of water when a limited number of underdrains are provided. For example, when using a filter bottom composed of stacked bricks with open joints (10 mm wide), four layers of gravel are normally used with the following size ranges: 0.4 to 0.6 mm; 1.5 to 2 mm; 5 to 8 mm; and 15 to 25 mm; each layer about 10 cm thick (IRC, 1981b). A photograph depicting this graded-gravel scheme is shown in Figure 8-31.

FIGURE 8-31

Graded-gravel Scheme for Slow-Sand Filter



[SOURCE: Huisman and Wood, 1974, p. 58]

5) Underdrain system -- The simplest method of underdrainage consists of a system of main and lateral drains made from perforated pipes of asbestos, cement, or plastic. A filter bottom of stacked bricks or concrete slabs may also be used. Both types of underdrain system are shown in Figure 8-32.

6) Depth of filter box -- The minimum depth of the filter box is determined from the following elements (Paramasivam and Mhaisalkar, 1981):

Freeboard above supernatant level	0.20 m
Supernatant water	1.00 m
Filter medium (initially)	1.00 m
Four-layer gravel support	0.30 m
Brick filter bottom	0.20 m
TOTAL	2.70 m

It is general practice to use a filter box 3 to 4 meters deep, but a depth of 2.70 meters will reduce construction cost without sacrificing filter efficiency.

7) Number of filter beds -- At least two filter units always should be built, and reserve units should be provided for large treatment plants. Table 8-10 gives some rough guidelines for determining the number of filter units for a given design population (Arboleda, 1973).

8) Filter control -- Slow-sand filters are operated conventionally at a constant rate. The rate is controlled by maintaining a constant head loss across the filter. A hand operated valve preceded by a venturi meter can be used to regulate the filtration rate and depth of water over the

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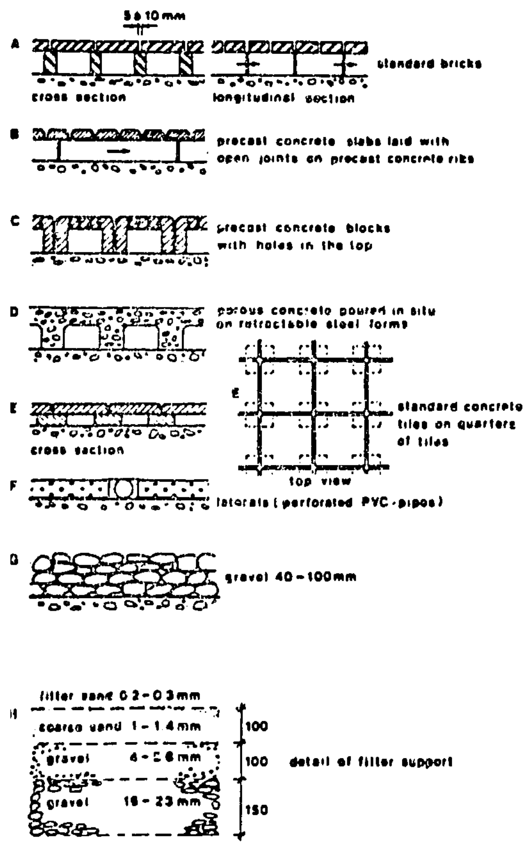
FIGURE 8-32

Different Types of Underdrain Systems for Slow Sand Filters



[SOURCE: Thanh and Hettiaratchi, 1981]

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TABLE 8-10: General Guidelines for Determining the Number of Slow Sand Filters Required for Different-sized Communities

POPULATION	TOTAL NUMBER OF UNITS	RESERVE UNITS
>2000	2	100%
2,000 - 10,000	3	50%
10,000 - 60,000	4	33%
60,000 - 100,000	5	25%

[SOURCE: Arboleda, 1973, p. 449]

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filter (see Figure 8-30). The normal range of head loss from clean to clogged conditions in the sand bed and filter appurtenances is 0.6 to 1.2 meters. An effluent weir is a valuable device to prevent negative head loss and air binding. On the other hand, the weir and control valve can be replaced by a simple unit consisting of a pair of telescopic tubes, the inner of which can be raised and lowered to adjust the rate of filtration, as shown in Figure 8-33.

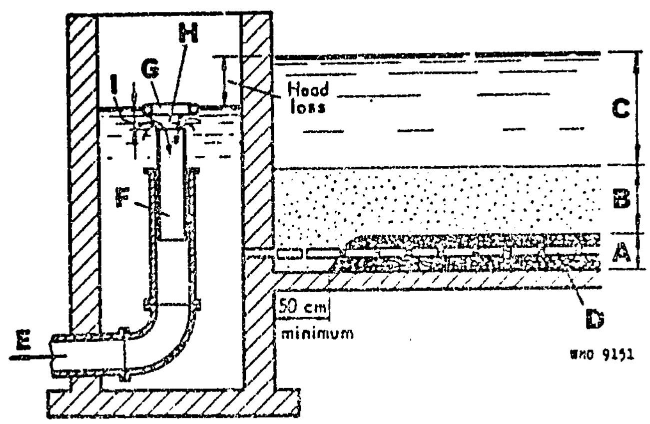
Consideration should be given to the possibility of operating the filter at a continuously declining rate. This is the case when the operator closes the raw water inlet, but keeps the filter outlet valve open. Then, the supernatant will drain through the filter at a continuously declining rate. The effluent weir should be set at least 0.2 meters above the top of the filter bed, to prevent damage to the "schmutzedecke" at the end of a declining-rate filtration period. Also, a sufficient quantity of water is required above the filter bed for storage. This type of operation may be applied during the night and allow for savings on manpower and capital investment costs (Thanh and Hettiaratchi, 1982).

Criteria for the design of slow-sand filters are summarized in Table 8-11.

In areas where sand is expensive or difficult to obtain, the surface scrapings from a slow sand filter may be

FIGURE 8-33

Telescopic-pipe Filtered Water Outlet



A = Depth of gravel and stone
B = Filter sand
C = Depth of water over filter bed
D = Under-drains
E = Filtered water
F = Telescoping pipe
G = Float
H = Circular weir
I = Constant head

Maximum allowable loss of head equals depth of water on filter.

[SOURCE: Wagner and Lanoix, 1959, p. 177]

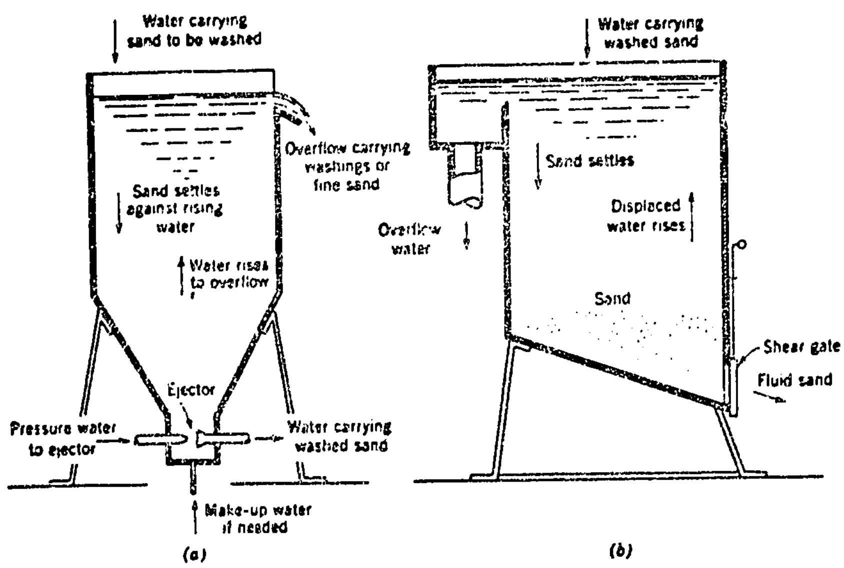
TABLE 8-11: General Design Criteria for Slow-Sand Filters

PARAMETER	RANGE	PREFERRED VALUE
Filtration velocity (m/h)	0.1 - 0.2	0.1
Depth of filter bed (m)	1 - 1.4	1.0
Area per filter bed (m^2)	10 - 100	---
Height of supernatant water (m)	1 - 1.5	1.0
Depth of system of underdrains (m)	0.3 - 0.5	0.4
Specifications of filter bed	UC = 1.5 - 3.0	
	E.S. = 0.15 - 0.35 mm	
Number of filters	minimum of 2	

[SOURCE: Thanh and Hettiaratchi, 1982, p. 33] washed, stored, and reused at a later date. However, the scrapings must be washed immediately, otherwise the material may go anaerobic, yielding taste and odor-producing substances that are nearly impossible to remove during any later washing processes (Huisman and Wood, 1974). The hydraulically-operated device, shown in Figure 8-34, can be used to wash sand removed from a filter. It functions essentially as an upward-flow clarifier; hence from a theoretical standpoint, the rate of overflow of the washer should not exceed the settling velocity of the smallest particle to be retained. However, in practice, turbulence and sand concentration reduce the desired rate of overflow appreciably so that the rate of flow of the incoming sand-water solution is generally sufficient to effect separation without supplementary water. The sand or grit that settles to the bottom is ejected hydraulically or can be removed by means of a shear gate. Pipes carrying sand-water solutions should be sized for velocities of 1.5 m/sec or higher. About 8 m^3 of sand per hour can be washed per square meter of washer surface area (Fair, Geyer, and Okun, 1968). For small installations, the sand washer (equipped with a shear gate) can serve as a storage area for cleaned sand; for larger installations a sand separator (Figure 8-34) can be used to effect separation of the sand from the washing water and for storage. An adaptation of the foregoing sand washer has been used successfully for

FIGURE 8-34

a) Hydraulically-operated Sand Washer
b) Gravity-operated Sand Separator



[SOURCE: Fair, Geyer and Okun, 1968, vol. 2, pp. 27-35] years at the Madras waterworks in India (Huisman and Wood, 1975).

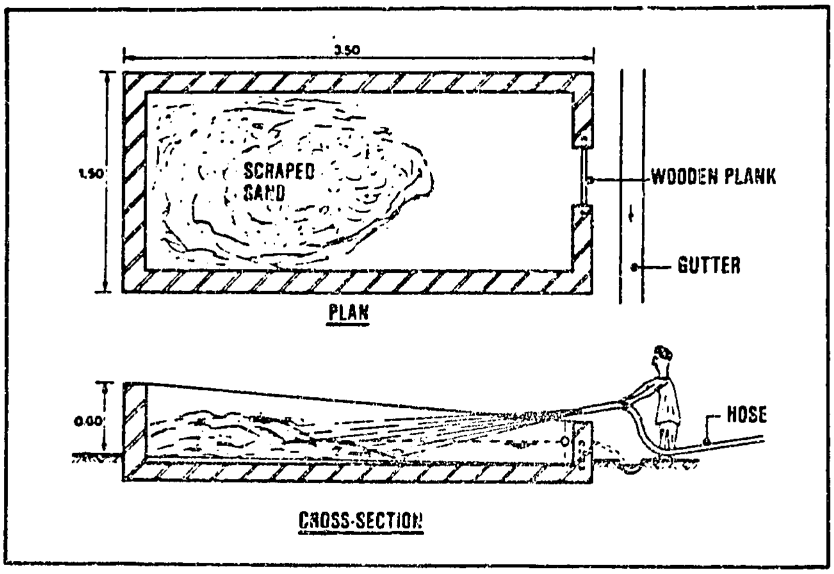
In smaller plants, where hydraulically-operated sand washers are not practicable, sand may be washed entirely by hand as illustrated in Figure 8-35. The sand is agitated in a box with water running through it at a low velocity so as not to wash out the fine particles. This process continues until the washing water clears, indicating that the sand is clean. The sand can then be stored and is ready for replacement on the filter.

When the bed reaches a minimum thickness of 0.5 to 0.8 m, the bed should be resanded. The clean sand should be placed on top of the gravel, and the older sand should be placed on top to provide seeding with microorganisms to form the "schmutzdecke" more rapidly and to assure that all the sand will be cleaned from time to time.

A two year study carried out for the Oxfam relief agency by Imperial College, London, England, is looking at the feasibility of using synthetic mats to cover the sand bed of slow sand filters and theoretically permit easy cleaning (Nigel Grahan, Department of Civil Engineering at Imperial College, personal communication). The mat is made of Bondina, a material normally used in air conditioning, which is characterized by very small interstices, and provides a suitable environment for the growth of microorganisms to form the "schmutzedecke." When the mat

FIGURE 8-35

Washing Platform for Manual Cleaning of Sand



[SOURCE: IRC, 1981b, p. 264]

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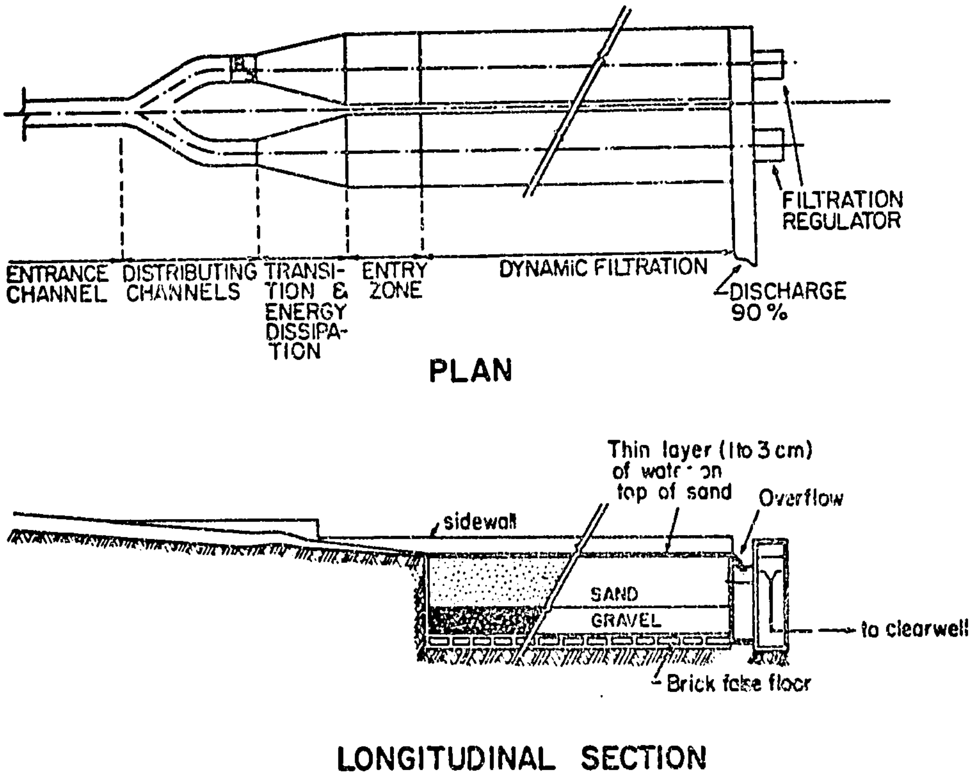
becomes clogged, it can be removed manually from the sand bed, cleaned by pressurized water, and replaced on the bed. This procedure replaces the more tedious method of scraping the top 1 to 3 cm of sand from the bed. Also, the ripening period that is required for the formation of the "schmutzedecke" is thought to be shorter with synthetic mats as compared to that for the sand bed alone. Field-testing of this new technology is underway in Honduras.

Dynamic Filtration

A unique type of slow-sand filtration system, called dynamic filtration, has been used in several rural areas of Argentina (Arboleda, 1973). Basically, the filter consists of a shallow channel, about 1 meter in height, with a sand bed and underdrain system similar to those used in conventional slow-sand filters (see Figure 8-36). The raw water flows over the bed surface with a velocity of 0.25 to 0.35 m/sec, forming a thin fluid layer (about 1 to 3 cm) and then over a weir into an overflow channel. At the same time, part of the flow (about 10% of the total flow) percolates through the sand bed, into the underdrain system, and is conveyed to a clear well at the same rate as in a conventional slow-sand filter. The main advantages of this type of filtration system is the very low construction cost, as the height of the filter walls are very low and may be built of unreinforced concrete or brick masonry. Also, in contrast to conventional slow-sand filtration, turbidities

FIGURE 8-36

Dynamic Filtration - Argentina



[SOURCE: Arboleda, 1973, p. 459]

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greater than 50 NTU can be applied, since the top of the sand bed is continuously cleansed by the relatively high velocities of the water passing over it, thereby reducing clogging problems. The main disadvantage is that the "schmutzedecke" formed in these filters is not as effective as those in conventional slow-sand filters, thus, the bacteriological removal is smaller. Of course, dynamic filters can only be used in places where water is abundant, i.e. the source of supply must be at least 10 times the capacity needed.

Information Sources on Slow-Sand Filtration

Information on the design and operation of slow-sand filters is contained in two comprehensive design manuals by Huisman and Wood (1974), and the International Reference Center (1978). The former looks the design, construction, and operation of modern slow-sand filters, the theory of biological filtration and the various methods of cleaning filters, which range from simple manual techniques to advanced mechanical or hydraulic systems. The latter focuses on simple designs for developing countries, and includes guidelines for the design and construction of small slow-sand filters, suitable treatment methods for the removal of turbidity, as well as four typical designs with capacities between 25 to 960 m^3/day. Complete plans and specifications for these designs are also included. Arboleda (1973) discusses pertinent design criteria and

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simple flowrate controllers for three types of slow sand filters, viz. conventional, upflow and dynamic filters. The IRC design manual, Small Community Water Supplies (IRC, 1981b), devotes an entire chapter to the design and construction of slow sand filters for small communities in developing countries and includes illustrations of several simple types of designs.

Economic considerations in the design of slow sand filters, together with basic design criteria, are given by Paramasivan and Mhaisalkar (1981). Mathematical models are developed which optimize the number and dimensions of filters in order to minimize filter costs (see Chapter 10, "Construction Costs of Water Treatment Plants").

A comprehensive annotated bibliography on the subject of slow sand filtration has been published by the IRC (1977). The selected references mainly deal with the technical aspects of the process. An author and key word index is provided as well as a list of institutions and organizations that can provide further information on the subject.

IX. MODULAR AND PACKAGE DESIGNS FOR STANDARDIZED WATER TREATMENT PLANTS

The conventional engineering approach for water treatment plant design involves planning on an individual community basis, or regional planning when several communities are to be served by a single project, followed by preliminary engineering and detailed engineering designs. However, this approach does not lend itself to the swift construction of a large number of small projects, which is the situation commonly encountered in the developing countries, particularly those committed to the International Water Supply and Sanitation Decade. One alternative is to adopt standardized procedures for the planning, design, and construction of water supply elements in order to decrease the time needed for the design of projects, reduce the number of experienced designers needed, and lower the overall cost. Other advantages which accrue from the reasonable use of standard designs include (Brown and Okun, 1968; IRC, 1981):

1) Expansion of the productivity of the skilled engineering designer;

2) Simplification of the design problem for the experienced engineer or technician;

3) Reduction in the cost of detailed design, thereby allowing more money to be spent on preliminary studies which are often slighted;

4) Reduction in construction costs if standards are based on the use of local materials;

5) Improvement in construction quality;

6) Simpler operation training;

7) Lower maintenance costs as spare parts could be more easily stocked; and

8) Promotion of local industry and expertise in the manufacture of equipment at low cost.

The main disadvantage in adopting standard designs is that they may become rigid and inflexible, thereby tending to inhibit the imaginative engineer and hence stifle improvement. This could be a serious problem if standards do not permit or encourage innovation.

Standard design manuals that are written for a particular country should reflect that country's unique conditions and needs, although the experiences of other countries or the work of international agencies may be helpful. For example, the design manual on slow-sand filtration published by the IRC (1978), the manual on modular water treatment plants produced by CEPIS (1982), or the standard designs presented in this manual (e.g., upflow-downflow filters described in Chapter 8), could readily be incorporated into a country-specific design manual. In general, standard designs and specifications should be kept simple, keeping in mind the need for quick installations at low cost, and ease of construction, operation and maintenance, while still providing the minimum acceptable level of service.

In Latin America, a "systems" approach has been developed for the promotion, design, construction, and operation of water and sanitation projects for small communities (Donaldson, 1976). Under this approach, projects are broken down into their component parts and each is studied for its effect upon the others. These elements are then coordinated to yield the lowest cost solution that meets the desired goals, i.e. the implementation of the greatest number of systems in the shortest time. For example, the technical aspects of a project are designed using existing maps or aerial photographs, standardized design criteria (e.g. 200 liters per capita per day, etc.), predesigned elements (modular treatment units, pump houses, etc.), and standardized equipment lists. The materials are gathered in a central place and sent to the community as a package together with any necessary tools or equipment not available locally. Also, professionals are available to involve the local community in the project, including the training and supervision of workers. Using this approach it is possible to delegate a considerable amount of work to intermediate level technicians and local workers.

Two types of standardized designs that have been used for treating water in small communities are: package plant designs; and modular plant designs. For the purposes of this manual, package plant designs refer to compact treatment units, generally made of steel, manufactured entirely in the factory, and transportable to remote areas; whereas modular plant designs refer to compact treatment units, generally made of concrete or masonry, and assembled either partly or entirely on-site without large or complicated equipment. The compact nature of both types of plants may be attributed in part to technological advances, such as gravel-bed and heliocoidal-type flocculators, inclined-plate settling, and dual-media filters that, when properly designed, can greatly reduce the size of the treatment units. However, using such technologically advanced units at higher loadings requires skilled operation. Often, simple standardized units should be used at low loadings to assure proper operation and reliability. The merits of package and modular types of designs for developing countries, as well as some typical plant layouts and design criteria, are presented below.

Package Water Treatment Plants

The popularity of package plants in the industrialized countries has grown in recent years, stimulated by rising construction and labor costs of custom-designed treatment facilities, particularly for smaller installations. Also, savings are realized in operational costs, as such plants are often automatic and designed for virtually unattended operation. Package plants are preassembled in factories where costs can be more carefully controlled than in the field. In most designs, on-site assembly and installation requirements are kept to a minimum. Accordingly, package plants have become a practical and economical solution for water treatment in small communities in North America and Europe.

In contrast, conventional package water treatment plants are not as well-suited for small communities in developing countries compared to facilities constructed wholly or partly on site because:

(1) Low-cost local labor is available in most communities and hence on-site construction costs are low as compared to those encountered in the industrialized countries.

(2) On-site construction can provide additional jobs for the local community, and, concomitantly, instill a sense of ownership to those that contribute their time and effort towards the project. This process encourages better operation and maintenance than does a package unit installed by outside contractors.

(3) The use of steel package plants in humid tropical countries in conjunction with corrosive chemicals (e.g.

alum, hypochlorites) requires special attention to preventive maintenance.

(4) Some developing countries do not have the technical capability nor supporting infrastructure to manufacture and maintain package plants. The economy of scale for manufacturing most types of package plants demands a large-scale operation in order to procure the necessary materials, manufacture several types of plants, transport and erect the plants, and establish a proper operational level. The importation of package plants from foreign proprietors is not likely to be economically feasible, as such units are expensive and overly mechanized, and when repairs are necessary, leave the user completely dependent upon spare parts from abroad.

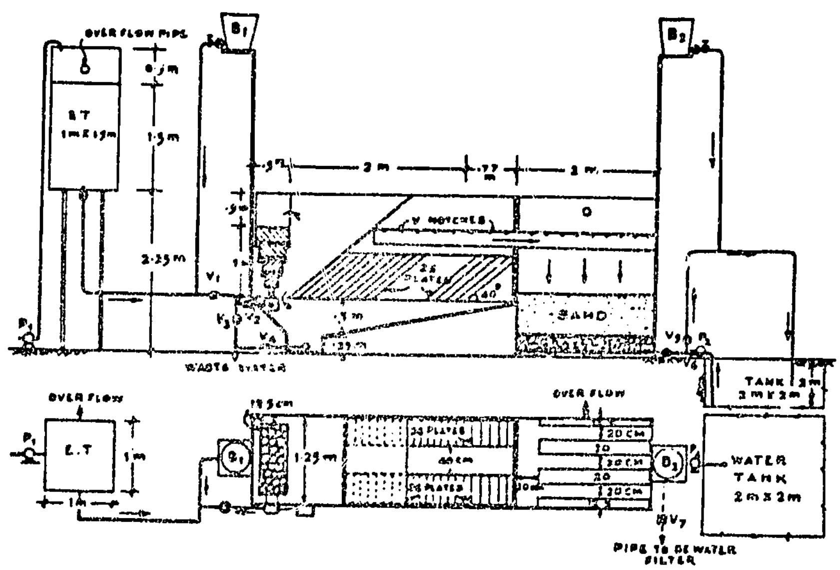
Simple package water treatment plants, manufactured inside the country, may be practical in places where a large number of small treatment facilities are needed, or where local conditions are unfavorable for on-site construction (e.g. lack of construction materials or low-cost labor, poor terrain or soil conditions). Bhole (1981) has suggested that package plants for rural areas in developing countries fulfill the following requirements. The plants should be (1) sturdy; (2) simple to operate and provide easy access to any of their parts; (3) reliable; (4) requiring only minimal mechanical equipment and running costs; (5) able to operate without electrical energy; (6) low cost; (7) easy to transport and install with minimum construction work at site; and (8) able to treat surface water.

Bhole has designed a simple package plant, taking into account the above mentioned criteria. A detailed plan and elevation of the plant is shown in Figure 9-1. The plant is comprised of the following: (1) an alum dosing unit, consisting of a large size plastic bucket for storing alum solution; (2) a gravel-bed flocculator, consisting of sections of increasing cross-sectional areas to produce tapered velocity gradients (described in Chapter 6, "Gravel-Bed Flocculators"); (3) an inclined-plate settling tank consisting of 26 plates located below a V-notched weir that conveys the settled water to the filter unit; (4) a filter unit, consisting of sand and supporting gravel media, and a perforated pipe underdrain system; and (5) a chlorination unit, similar to the alum unit but containing a solution of bleaching powder.

Raw water is transmitted by a diesel pump (P_1) to the elevated tank (ET) which provides the necessary head for gravity flow through the treatment unit. The second pump (P_2) is used for cleaning the filter and settling tank. During filtration, valves V_1, V_2, and V_5 are opened and valves V_3, V_4, and V_7 are closed. During backwashing, valves V_4 and V_6 are opened and valves V_1, V_2, V_3, V_5, and V_7 are closed and pump P_2 is started. The washwater flows upward through the filter, collects in the troughs, and is

FIGURE 9-1

Package Water Treatment Plant - India



[SOURCE: Bhole, 1980, p. 320]

then taken to the settling tanks where it is drained via a floor drain. To drain the flocculator unit, valves V_1 and V_4 are closed and valves V_2 and V_3 are opened.

The dimensions of this package plant are 5.3 x 1.25 x 1.25 meters, it weighs 1.3 tons, and has a capacity of 270 m^3/day, which is sufficient for a population of 2000 people, assuming a per capita consumption of 45 liters per day. Several package plants can be operated in parallel to meet larger water demands. The cost of the plant is about Rs. 20,000 (US$2500), but could be reduced if it is manufactured in large numbers. Figure 9-2 shows the "packaged" appearance of this plant, and pertinent installation criteria and process capabilities.

A steel package plant designed specifically by APS Technical Services, Ltd., (1982) in England for developing country applications is comprised of a single module containing hydraulic flocculation, inclined-plate settling, and rapid sand filtration. The module itself is a standard 6-meter shipping container which can be handled by any port or railroad which takes conventional containers, and it will fit any container ship, train, or truck. By making the containers part of the plant, the problems and costs of packing for shipping and carrying tanks within containers are overcome, while achieving a known shipping cost to almost any destination in the world. A flow diagram of the plant is shown in Figure 9-3. The rapid mixer is comprised of a

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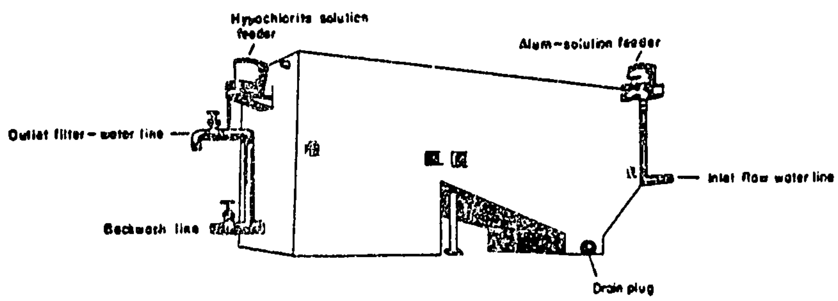
FIGURE 9-2

Isometric View of Indian Package Plant,
Together with Installation Requirements and Process Capabilities



CAPACITY

Raw water processing rate	270 m^3/day
Inlet raw water line	75 mm dia
Outlet filter water line	75 mm dia
Drain plug	75 mm dia
Backwash line	75 mm dia
Gun metal valves	
Raw water	75 mm dia
Filter water	75 mm dia
Backwash	75 mm dia
Overflow	25 mm dia
Retention tank	18 m^3 (recommended)

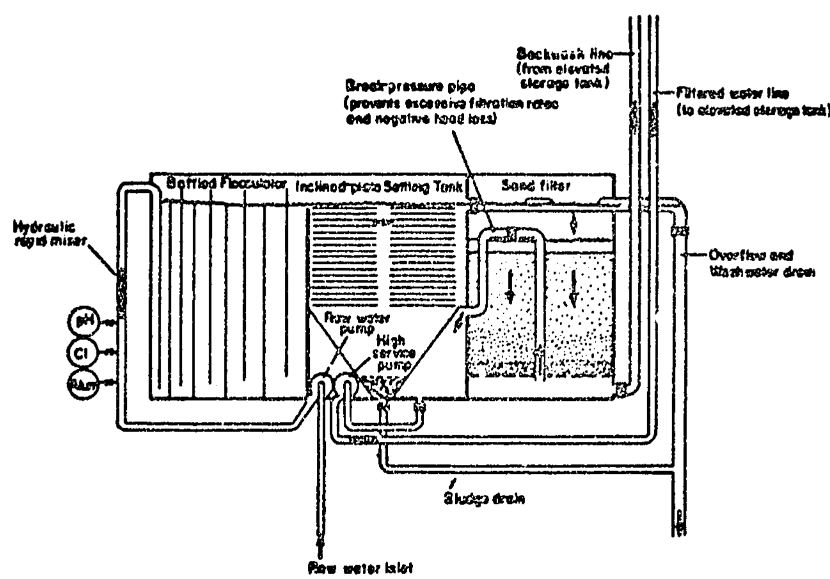
PROCESS CAPABILITY

Maximum turbidity handled ~ 500 units
Turbidity of treated water ~ 10 units
Alum dosing for rapid flocculation
Chlorination to reduce bacterial contamination

[SOURCE: UNC International Programs Library, personal communication]

FIGURE 9-3

Flow Diagram of Steel Package Plant
Manufactured in England



[SOURCE: APS Technical Services, Ltd., 1982] short length of pipe enclosed by wire screening at both ends and filled with short pieces of small-diameter plastic pipes that serve to agitate the water passing around and through them; the flocculator is comprised of lightweight plastic baffles that can be removed easily for cleaning; the settling tank contains parallel plastic plates inclined at 60^{o} from the horizontal through which the flocculated water flows horizontally; and the filter box contains a 60 cm layer of sand supported by gravel, and a main and lateral underdrain system. The underdrain system of the filter is connected to a backwash line and a break-pressure pipe, the latter of which is used to avoid both excessive filtration rates when the sand bed is clean, and negative head losses or air binding in the sand bed. All chemicals (alum, chlorine, and lime) are presently added to the incoming water ahead of the rapid mixer, although it is possible to add chemicals at the discharge side of the high service pump. The chemical solutions are contained in flexible bags and drawn into the influent pipeline, in proportion to the flow rate, by the negative pressure created by an orifice placed upstream of the dosing points.

The technical specifications of the package plant are as follows:

1) dimensions: 6 m x 2.4 m x 2.4 m
2) dry weight: 13,000 kg
3) operating weight: 45,000 kg

4) *flow rate*: 360 to 530 m^3/day

5) *ground loading*: 75 kg/m^2

6) *tank construction*: lined carbon steel; and

7) *pumps*: electric motor or diesel.

An elevated storage tank is also provided by the manufacturers for backwashing the filter as well as providing pressure for distribution. The storage tank, depicted alongside the package plant in Figure 9-4, is self-elevating and can be erected by 4 to 5 laborers without the need for cranes or mechanical equipment.

Modular Water Treatment Plants

Water treatment plants based on modular designs may be more quickly built, while still allowing contributions from the local community such as raw materials, and involvement in construction, operation, and maintenance; hence, such plants are quite attractive for standardized water supply projects for small communities in developing countries. Modular designs that are standardized reduce the type and number of plant devices, thereby facilitating a more efficient system of procurement of spare parts, training of operators and ease of repairs. To further shorten the time span for project implementation, plants may be comprised of modular units that are prefabricated, and transportable to construction sites for final assembly. Although modular designs are amendable to either concrete or steel construction, concrete is generally preferred

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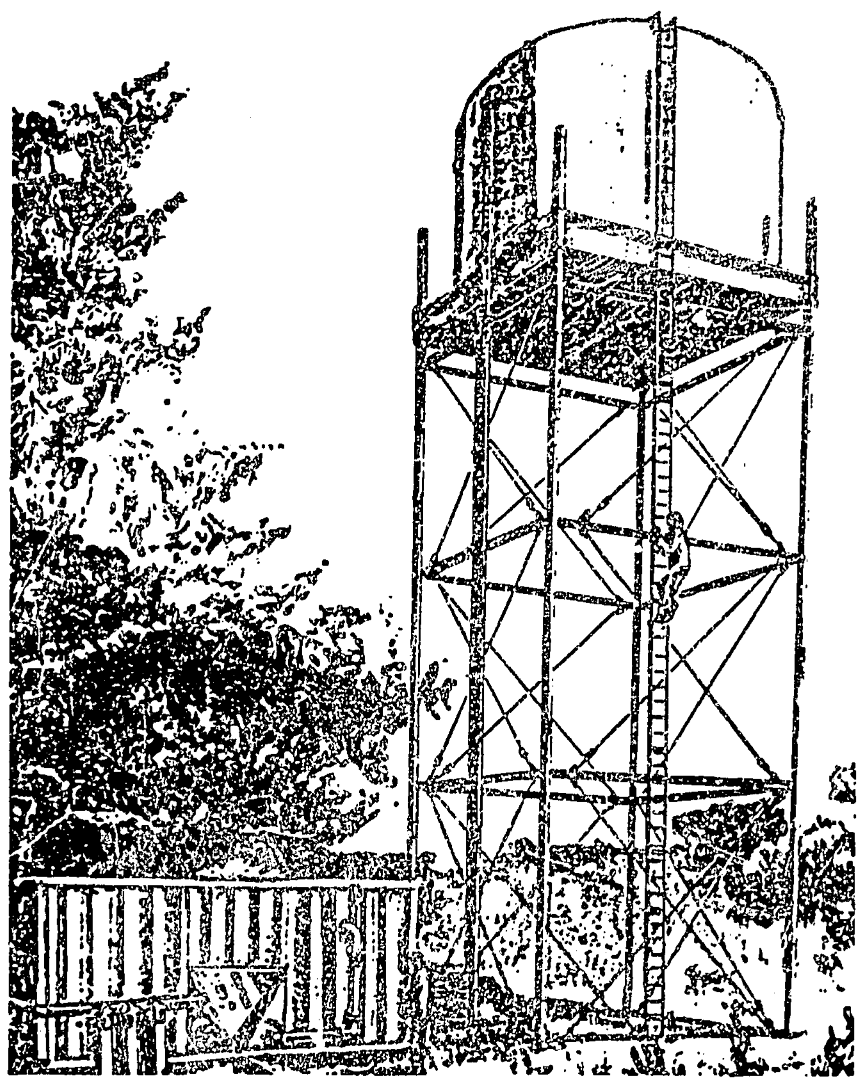
FIGURE 9-4

Steel Package Plant with Self-Elevating Storage Tank

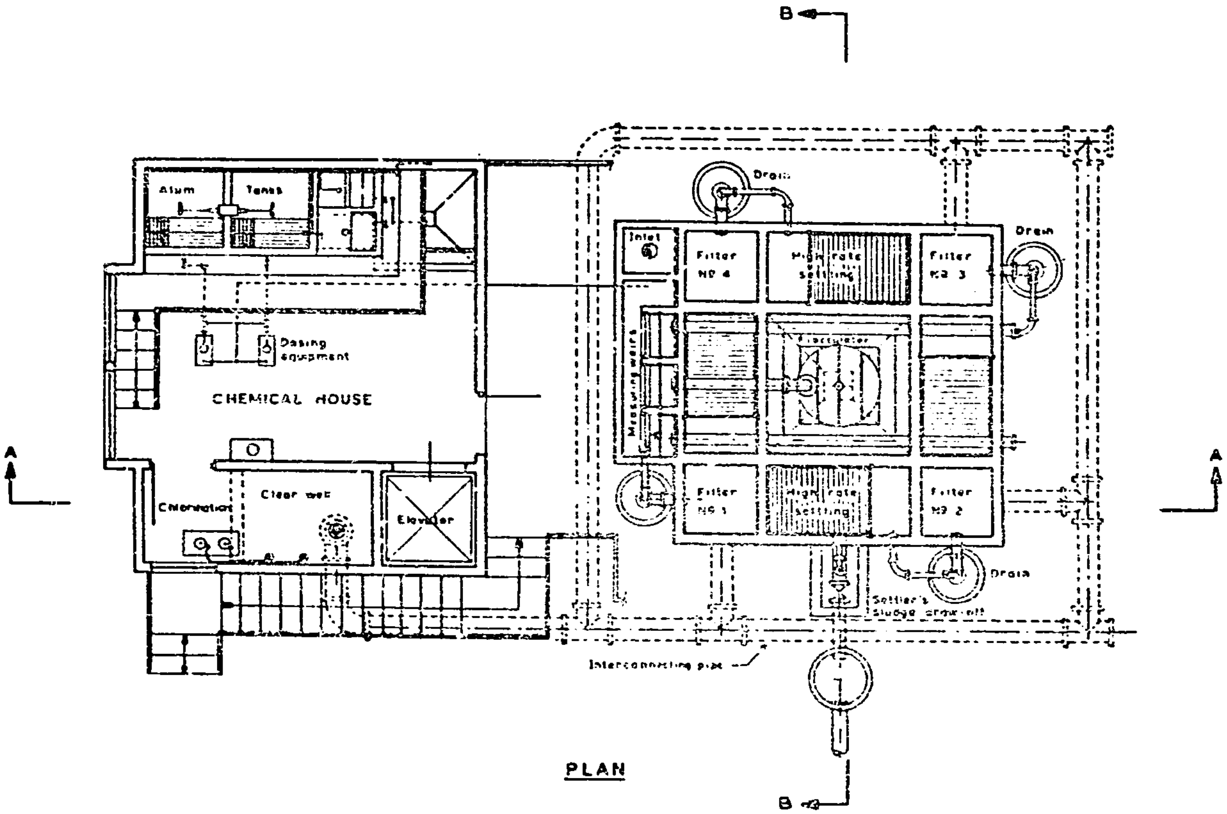


[SOURCE: APS Technical Services, Ltd., 1982] because of its wide availability in developing countries, comparatively low cost, and resistance to corrosion. Moreover, the majority of skilled and unskilled workmen employed in developing countries are more familiar and proficient with concrete construction than with steel. Two types of modular water treatment plants developed in Latin America and Indonesia are described below. In addition, the upflow-downflow filtration plants described in Chapter 8 and the various plants described in the CEPIS manual (1982) are suitable modular units for developing countries.

The water treatment plant serving the city of Prudentopolis, Brazil (population 7500) consists of a modular unit 4 meters in plan, having a capacity of 1000 m^3/day (Arboleda, 1976; Sperandio and Perez, 1976). The plant consists of a hopper bottom square tank with four 1x1 meter dual-media filters located at the corners, four 1x2 meter inclined-plate settling tanks placed near the outside walls, and, at the center, a flocculation chamber with four compartments. A plan and section of the plant are shown in Figures 9-5 and 9-6, respectively. The raw water enters a rapid mix chamber at one corner of the tank, where alum and lime are added. Agitation is caused by the discharge of the raw water through a circular weir. The water then enters a distribution channel, flowing over one of three triangular

FIGURE 9-5

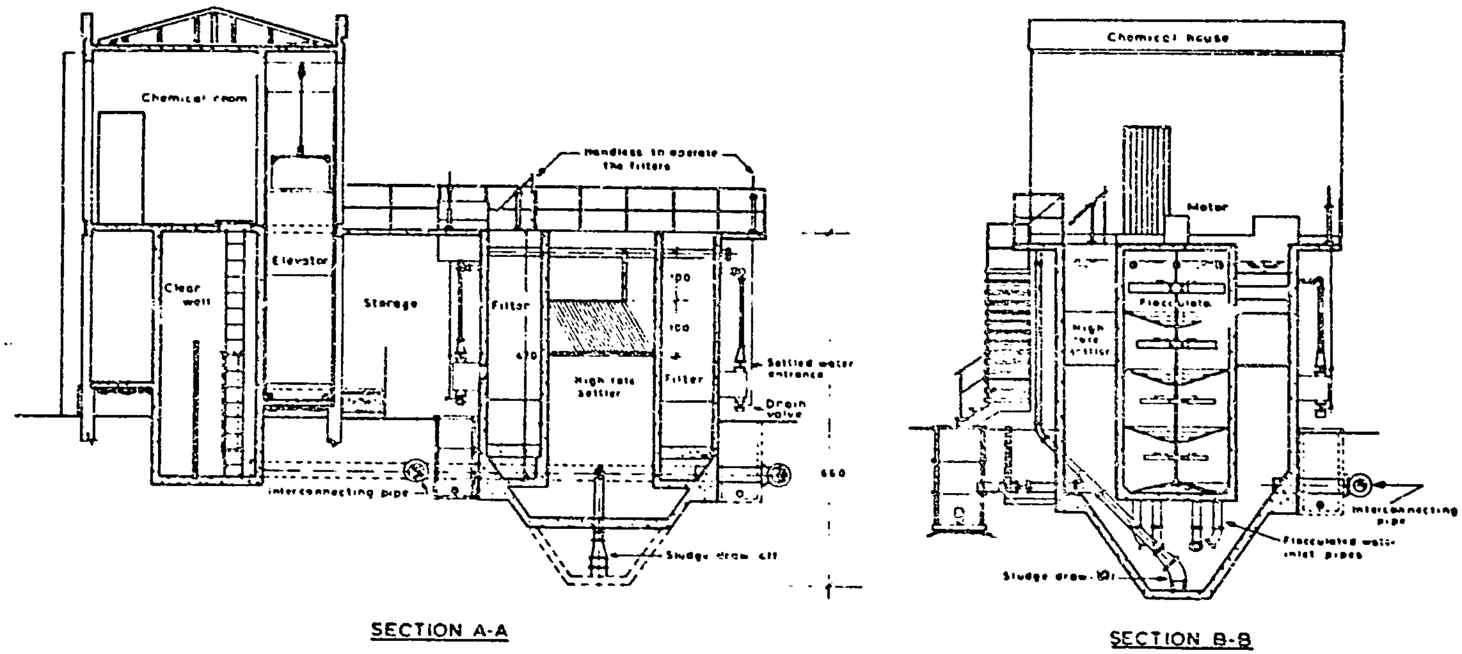
Modular Treatment Plant (1000 m^3/day) in Prudentopolis, Brazil (plan)



[SOURCE: Arboleda, 1976, p. 252]

FIGURE 9-6

Modular Treatment Plant (1000 m^3/day) in Prudentopolis, Brazil (sections A-A; B-B)



[SOURCE: Arboleda, 1976, p. 253]

weirs and conveyed via a cast-iron pipe to the flocculation chamber which is comprised of four vertically arranged compartments. Tapered mixing is provided by wooden paddles of different cross-sectional area that are driven by a single 370 watt (1/2 hp) electric motor. The flocculated water leaves the bottom chamber via six cast-iron pipes that discharge into four upflow settling tanks equipped with a series of 1x1 meter asbestos-cement parallel plates placed 5 cm apart. The settled water enters the filter via cast-iron pipes that are attached to a metal box located on the outside faces of the filter walls. The drain pipe is also attached to this metal box. Two butterfly valves, controlled by a single handle, can simultaneously close the filter influent pipe and open the drain pipe, or vice versa, to initiate either filtration or backwashing operations. The four filters are designed for interfilter backwashing, as they are all interconnected by a 300 mm cast-iron pipe (see Chapter 8 for information on the design of interfilter-washing units). To regulate the backwash flow, a sliding pipe placed in the clear well can be raised or lowered to decrease or increase the backwash rate.

An important feature of the plant, which should be included in any type of design, is the potential for expanding plant capacity. In this case, when two or three modules are to be used, the raw water influent flow is split by means of three triangular weirs installed on the side of

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the distribution channel. The two outside weirs discharge directly into cast-iron pipes which are used to convey the influent to two additional treatment modules. The total construction cost of the single-module Prudentopolis plant, including the chemical building, is $54,000 which compares favorably to that of a conventionally designed plant of the same capacity (between $81,000 and $110,000). A complete description of this plant, including a comprehensive technical and economic evaluation, are described in a report issued by the US Agency for International Development (Sperandio and Perez, 1976).

An extensive modular water supply program for rural communities in Indonesia was initiated in 1979 under the joint direction of the Indonesia Directorate of Sanitary Engineering and the IRC (IRC, 1981). The purposes of the program were three-fold: (1) to study a modular approach, i.e. using standard components for the planning and design of small water treatment plants for surface water in Indonesia; (2) to prepare criteria, specifications, and working plans for the planning and design of these modules for domestic manufacture; and (3) to study and to comment on existing designs in order to evaluate the economic aspects of the use of local building materials (concrete and steel) and to make recommendations accordingly.

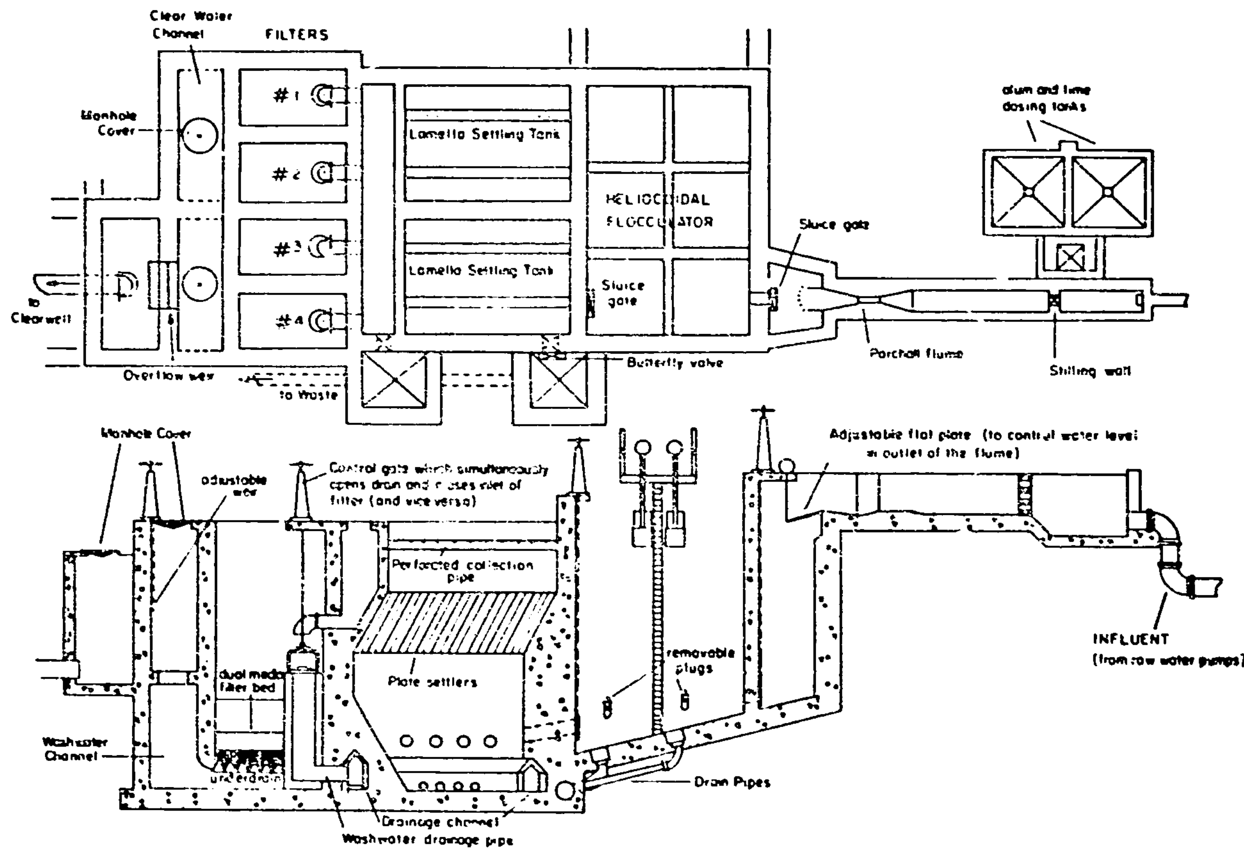
Standard designs were developed for both concrete and steel plants having capacities of 1730, 3460, 5190, and 6920

m^3/day. Figure 9-7 shows a 1730 m^3/day concrete plant comprised of: (1) rapid mixing by means of a hydraulic jump formed immediately downstream of a Parshall flume, which is also used as a flow measuring device; (2) flocculation in six square chambers where flow alternates between upflow and downflow (heliocoidal-flow type); (3) inclined-plate sedimentation using asbestos-cement plates inclined at 60°; (4) filtration with dual-media filtes having interfilter-washing capabilities; and (5) chemical feeding using simple constant-rate feeders for alum and hypochlorite dosing, and lime saturation towers. The above mentioned unit processes were employed in all of the concrete and steel plant designs, but the unit processes for the steel plants were designed particularly for prefabrication and transportability. Accordingly, the steel components were built no larger than 2x5x3 m, weighed no more than 4.5 tons, and could be readily transported to remote areas and quickly assembled at the construction site.

An international seminar held in Indonesia on modular approaches for water supply programs (IRC, 1981) recommended that the Indonesian designs be used as models in other developing countries with, if possible, intercountry field testing of the modular treatment plants. An unpublished report on the standard water treatment plants in Indonesia which includes general and detailed design criteria, descriptions of the concrete and steel plant designs, and

FIGURE 9-7

Modular Treatment Plant (1730 m^3/day) in Indonesia (plan; section)



[SOURCE: adapted from IRC, 1981a]

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the application of the standard designs for various types of surface waters is available from the IRC (IRC, 1980).

X. COSTS OF WATER TREATMENT PLANTS IN DEVELOPING COUNTRIES

Preliminary planning of water supply projects, including the final selection of treatment components and arrangements for financing, must be based on reliable cost data. Such data are difficult to obtain in developing countries, particularly in regions where water supply systems are to be built for the first time. In such instances, reasonable cost estimates for construction, operation, and maintenance may be obtained indirectly by using: (1) cost data for similar plants that have been built in other regions within the country or in another developing country with similar characteristics; (2) general cost curves that are based on the costs of a variety of plants constructed within the country; or (3) general predictive cost equations developed for similar situations. Although costs from one country are not generally directly applicable to other countries, the relationships among the experienced costs of the various types of treatment (e.g. conventional, direct filtration, package plants), and particularly the unit costs as a function of the size of the plant, are useful.

The purposes of the cost data and the predictive-cost equations presented in this chapter are: (1) to assist administrators, engineers, and public officials in developing countries, who are planning future water treatment schemes, to assess the general level of capital and recurrent costs as a tool for planning; (2) to allow officials to check whether cost estimates submitted by engineers are reasonable; and (3) to provide financial guidelines for making preliminary decisions on water supply schemes.

This chapter begins by discussing the general cost functions used widely by officials in the water supply fields; followed by sections on: (1) construction-cost curves and tables specific to countries in Asia and Latin America, and comparisons among these data; (2) operation and maintenance (O&M) costs and the inherent difficulty in estimating such costs; and (3) general predictive equations for construction and O&M costs developed specifically for rapid filtration and slow-sand filtration plants in Africa, Asia, and Latin America.

Unless otherwise stated, the cost data in this manual have been adjusted to the March 1982 Engineering News Record (ENR) Construction Cost Index of 3729 based on an ENR Index of 100 in 1907, so that all costs are on a common basis (Engineering News Record, March 1982). The ENR index is based upon the average cost, at a particular time, of constant quantities of structural steel, Portlant cement, lumber, and common labor in 20 cities in the United States.

Foreign currencies have been converted to US dollars using the July 1982 exchange rates listed below:

1)	Brazil (Cruzeiro)	170 Cr = US$1.00
2)	England (Pound)	0.58 L = US$1.00
3)	India (Rupee)	8.8 Rs. =US$1.00
4)	Thailand (Baht)	23 B = US$1.00

The General Cost Equation

The relationship between plant capacity and construction costs is often represented by the following power function which reflects the economies of scale present in large water projects:

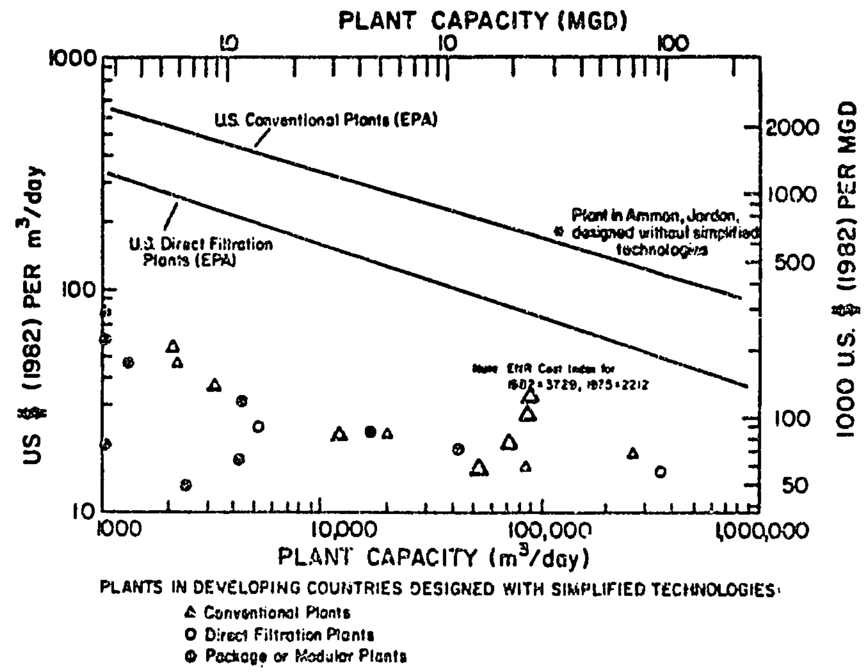
$$C = a\,Q^{b} \qquad (10\text{-}1)$$

where C = construction costs; Q = plant capacity; a and b = constants. This relationship is the basis for most of the cost curves presented in this chapter.

The constant b determines the manner in which cost changes with plant capacity. Large economies of scale are associated with small values of b. For example, the EPA cost curves for conventional and direct filtration plants constructed in the United States (see Figure 10-1) have b-values of 0.70 and 0.48, respectively; therefore, the construction costs in both cases will increase with capacity but the cost/unit capacity will decline. On the other hand, the construction cost equation for slow-sand filters in India (Eq. 10-2) has a comparatively high b-value of 0.86;

FIGURE 10-1

Comparative Construction Costs (1982 US$) of Conventional Filtration Plants in Developing Countries



hence, the economies of scale are not as great as those for US filtration plants. While optimum design values for new facilities depend upon many factors including expected rates of growth, discount rates, useful life of the facilities, and the ease of expansion, generally unless b-values are below 0.7, there is little economic incentive to overdesign plants (Paramasivam et al., 1981).

The constant a is equivalent to the construction cost of a plant with unit capacity, and is also a function of the ENR construction cost index. For example, the EPA cost equations for US plants, originally based on 1975 prices, have been adjusted to 1982 prices, as shown in Figure 10-1, by multiplying each of the a-values of the respective equations by 3729 (March 1982 ENR cost index), and dividing by 2212 (1975 ENR cost index). The component b remains unchanged in this updating procedure.

There are numerous factors that affect the costs of the various methods of water treatment, apart from plant capacity (Q) or basic construction costs (as reflected by the ENR construction cost index). Some of these factors include (1) the type of plant; (2) the local costs of materials and labor; (3) design criteria (conservative designs lead to larger components and higher costs); (4) geographical location; (5) transportation; (6) climatic conditions; (7) level of competition among building contractors; and (8) delivery time for critical items. A

TABLE 10-5: Construction Cost (1982 US$) for Optimum Number of Slow-Sand Filter Units in India[a] factor of great importance in developing countries is the cost of the equipment and materials that have to be imported. Obviously, it would not be feasible to incorporate all of these factors into a preliminary cost estimate for a particular project; nevertheless, any known conditions that would substantially affect the cost of a project should be considered, and appropriate adjustments made to the cost data.

Construction Costs of Water Treatment Plants

This section contains cost curves and tables for the construction of rapid and slow-sand filtration plants in developing countries. Brief descriptions and construction costs for plants described in this manual, and for other plants designed simply and economically, are summarized in Tables 10-1 and 10-2, respectively. Plants are characterized and grouped generally as conventional, direct filtration, and package or modular.

Table 10-2 does not show separate costs for the major treatment plant components but the following guidelines may be used as a rough estimate (Sanks, 1979):

COMPONENT	% OF TOTAL CONSTR. COST
- Earthwork, general site work, and yard piping	15-20
- Sedimentation and flocculation basins	20-30
- Filters and appurtenant systems	20-35

TABLE 10-1: Description of Simplified Water Treatment Plants
(*Plants described in this Manual)

PLANT LOCATION (reference)	PLANT TYPE[a]	YEAR OF CONSTR.	CAPACITY (m^3/day)	MGD	PLANT UNIT PROCESSES[b]
Barranquilla, Colombia (Arboleda, p.c.)	C	1982	86,400	22.8	WRM, MF, HFSIP, IFW/DMF
Becerril, Colombia (Arboleda, p.c.)	C	1982	3,500	0.92	PFRM, HFF, HFSIP, IFW/DMF
Cali, Colombia (Wagner, 1982)	C	1979	260,000	68.0	WRM, MF, HFSIP, RSF
Cali, Colombia (Arboleda, p.c.)	C	1982	86,400	22.8	WRM, MF, HFSIP, IFW/DMF
Cochabamba, Bolivia* (Arboleda, 1976)	C	1975	20,000	5.00	PFRM, ICF, HFSIP, IFW/DMF
La Paz, Colombia (Arboleda, p.c.)	C	1982	12,000	3.17	PFM, PWF, HFSIP, IFW/DMF
Manaure, Colombia (Arboleda, p.c.)	C	1982	2,160	0.57	PFRM, HFF, HFSWIP, IFW/DMF
Manizales, Colombia (Arboleda, p.c.)	C	1982	69,120	18.2	WRM, MF, HFSWIP, IFW/DMF
Oceanside, California* (MacDonald & Streicher, 1977)	C	1977	65,000	17.0	MJRM, BCF, HFS, IFW/DMF
Pereira, Colombia (Arboleda, p.c.)	C	1982	51,840	13.7	WRM, MF, HFSWIP, IFW/DMF
Parano, Brazil (Wagner, 1982)	C	1978	87,000	23.0	PFRM, MF, UFS, DMF
Piracicaba, Brazil (Azevedo-Netto, p.c)	C	1981	65,000	17.0	PFRM, MF, HFS, DMF
Sao Paulo, Brazil (Azevedo-Netto, p.c.)	C	1981	1,040	0.27	OPRM, VFBCF, UFSIP, IFW/DMF
Sao Paulo, Brazil (Azevedo-netto, p.c.)	C	1981	2,200	0.57	OPRM, VFBCF, UFSIP, IFW/DMF
Linhares, Brazil* (Sperandio & Perez, 1976)	D	1974	5,200	1.40	PFRM, CUSF

TABLE 10-1 (cont.) : Description of Simplified Water Treatment Plants
(*Plants described in this Manual)

PLANT LOCATION (reference)	PLANT TYPE[a]	YEAR OF CONSTR.	CAPACITY (m^3/day)	MGD	PLANT UNIT PROCESSES[b]
Brasilia, Brazil (Wagner, 1982)	D	1981	380,000	100	HRM, DF
Parana, Brazil (Azevedo-Netto, p.c.)	SF	1975	1,300	0.34	UCC, DPF
Colon, Costa Rica (Institute of Water Supply & Sewerage, p.c.)	SF	1979	5[illegible]	0.15	UCC, DPF
Ramtek, India (Kardile, 1981)	UD	1973	2,400	0.63	UGBF, DMF
Chandori, India* (Kardile, 1981)	UD	1977	1,000	0.26	UGBWTS, DMF
Varangaon, India* (Kardile, 1981)	UD	1977	4,200	1.10	GBF, UFSIP, DMF
India* (Bhole, 1981)	P	1981	270	0.07	GBRM, GBFl, UFIP, DMF
Parana, Brazil (Wagner, 1982)	P	1980 (not including chemical house)	830	0.23	GBRM, GBFl, UFSIP, DMPF
Parana, Brazil (Wagner, 1982)	M	1979	43,000	11	OPRM, MF, UFSIP, IFW/DMF
Parana, Brazil (Wagner, 1982)	M	1979	17,000	4.5	OPRM, MF, UFSIP, IFW/DMF
Parana, Brazil (Wagner, 1982)	M	1979	4,400	1.2	OPRM, MF, UFSIP, IFW/DMF

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TABLE 10-1 (cont.) : Description of Simplified Water Treatment Plants
(*Plants described in this Manual)

PLANT LOCATION (reference)	PLANT TYPE[a]	YEAR OF CONSTR.	CAPACITY (m^3/day)	MGD	PLANT UNIT PROCESSES[b]
Parana, Brazil (Wagner, 1982)	M	1980	830	0.23	GBRM, GBFl, UFSIP, DMPF
Prudentopolis, Brazil* (Arboleda, 1976)	M	1975	1,000	0.26	OPRM, MF, UFSIP, DMF

[a]C = conventional rapid filtration; D = direct filtration; M = modular rapid filtration; P = package plant; SF = superfiltration; UD = upflow-downflow

[b]BCF = baffled channel flocculation; CUSF = contact upflow sand filtration; DMF = dual-media filters; DMPF = dual-media pressure filters; DPF = downflow polishing filter; GBF = gravel bed filter; GBFl = gravel bed flocculation; GBRM = gravel bed rapid mix; HFF = helicoidal flow flocculator; HFS = horizontal flow sedimentation; HRM = hydraulic rapid mis; IFW = interfilter washing; IP = inclined plates; MF = mechanical flocculators; MJRM = multijet rapid mix; OPRM = orifice plate rapid mix; PFRM = Parshall flume rapid mix; PWF = Pelton wheel flocculator; RSF = rapid sand filter; TS = tube settler; UCC = upflow contact clarifier; UFS = upflow sedimentation; UGBF = upflow gravel bed filter; VF = vertical flow; WRM = weir rapid mix.

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TABLE 10-2: Construction Costs of Simplified Water Treatment Plants
(*Plants described in this manual)

LOCATION/SOURCE OF PLANT (reference)	YEAR OF CONSTR	TOTAL CONSTR. COSTS	1982 CONSTR. (US$/$m^3$/day)	COSTS PER UNIT CAPACITY (US$/MGD)
Barranquilla, Colombia (Arboleda, p.c.)	1982	US$2,985,000	34	130,000
Becerril, Colombia (Arboleda, p.c.)	1982	US$120,040	34	130,000
Cali, Colombia (Wagner, 1982)	1979	US$3,800,000	18	70,000
Cali, Colombia (Arboleda, p.c.)	1982	US$2,354,000	27	100,000
Cochabamba, Bolivia* (Arboleda, 1976)	1975	US$260,000	22	83,000
La Paz, Colombia (Arboleda, p.c.)	1982	US$265,000	22	84,000
Manaure, Colombia (Arboleda, p.c.)	1982	US$109,440	50	190,000
Manizales, Colombia (Arboleda, p.c.)	1982	US$1,446,150	21	79,000
Oceanside, California* (MacDonald & Streicher, 1977)	1977	US$3,700,000	82	310,000
Pereira, Colmbia (Arboleda, p.c.)	1982	US$881,700	17	64,000
Parana, Brazil (Wagner, 1982)	1978	US$1,000,000	16	59,000
Piracicaba, Brazil (Azevedo-Netto, p.c.)	1982	US$5,000,000	77	290,000
Sao Paulo, Brazil (Azevedo-Netto, p.c.)	1981	US$79,000	80	300,000
Sao Paulo, Brazil (Azevedo-Netto, p.c.)	1981	US$97,000	47	180,000
Linhares, Brazil* (Sperandio & Perez, 1976)	1974	US$69,000	24	91,000

TABLE 10-2 (cont.) : Construction Costs of Simplified Water Treatment Plants
(*Plants described in this Manual)

LOCATION/SOURCE OF PLANT (reference)	YEAR OF CONSTR	TOTAL CONSTR. COSTS	1982 CONSTR. (US$/$m^3$/day)	COSTS PER UNIT CAPACITY (US$/MGD)
Brasilia, Brazil (Wagner, 1982)	1981	US$5,300,000 (design estimate)	15	56,000
Parana, Brazil (Azevedo-Netto, p.c.)	1975	US$36.000	47	180,000
Colon, Costa Rica (Institute of Water Supply and Sewerage, p.c.)	1979	US$10,000	22	83,000
Ramtek, India* (Kardile, 1981)	1973	US$16,000	13	50,000
Chandori, India* (Kardile, 1982)	1980	US$19,000	20	77,000
Varangaon, India* (Kardile, 1981)	1977	US$50,000	17	65,000
India* (Bhole, 1981)	1982	Rs20,000	10	39,000
Parana, Brazil (Wagner, 1982)	1980	US$27,000	38	140,000
Parana, Brazil (Wagner, 1982)	1979	US$650,000	19	71,000
Parana, Brazil (Wagner, 1982)	1979	US$320,000	23	88,000
Parana, Brazil (Wagner, 1982)	1979	US$140,000	31	140,000
Parana, Brazil (Wagner, 1982)	1979	US$76,000	55	210,000
Prudentopolis, Brazil* (Arboleda, 1976)	1975	US$35,000	59	23,000

- Operations and administration buildings	10-20
- Miscellaneous chemical tanks, small structures	10-15

The costs of the clearwells are not included because of their highly variable capacities, which depend on local circumstances.

Cost data from Table 10-2 are plotted in Figure 10-1 for (1) conventional rapid filtration plants; (2) direct filtration plants; and (3) modular and package plants; all of which have been designed with practical, low-cost technologies in developing countries. Also, cost curves developed by the EPA for US plants are shown in the figure for comparative purposes (EPA, 1978). These curves, originally based on 1975 cost data, have been adjusted to 1982 dollars. The costs for the simplified plants are about one order of magnitude lower than those for plants designed in the United States. For example, the conventional rapid filtration plant for the city of Cochabamba, Bolivia has a capacity of 20,000 m^3/day (5 MGD) and was built at a unit cost of US $22 per m^3/day (US $83,300 per MGD); while the unit cost of an identically-sized plant built in the United States is about US $260 per m^3/day (US $984,000 per MGD) - 12-fold larger. Plants designed without using simplified technologies in developing countries, however, may have capital costs even higher than those in the United States; as evidenced by plant under construction in Amman, Jordan (121,000 m^3/day or 32 MGD) which was designed with conventional technologies and has a total construction cost estimate of $22,100,000 (Wagner, 1982b). However, in most instances, it can be expected that conventional plants built in developing countries will cost less than similar plants in the US. Kardile (1981) has shown only 50% cost reduction between conventional and simplified plants in Iran. Even conventional plants in most developing countries will be simpler than plants in the US.

Plant upgradings can also be accomplished using simplified technologies at a cost of about US$14 to US$16 per m^3/day for capacities ranging from 40,000 to 60,000 m^3/day, which represents from 30 to 40% of the cost of expanding a plant using conventional technologies (Sperandio & Perez, 1976).

Part of the cost difference between plants designed in the US and the simplified plants in the developing countries can be explained by the lower labor costs and lower subsidized interest rates in developing countries; although these may be partially offset by expatriate contractors' high overhead costs (Wagner, 1982). The primary reason, however, lies in the approach to design of the simplified plants; emphasizing low-cost non-mechanized solutions that are compatible with socioeconomic and technical conditions in developing countries.

A cost study was conducted for the Brazilian State of Parana based on construction costs of eight water treatment

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plants having capacities from 2,000 to 45,000 m^3/day (SANEPAR, 1979). The resulting cost curve is shown in Figure 10-2. The plants were similar to the modular plant shown in Figures 9-3 and 9-4, consisting of hydraulic mixing and flocculation, inclined-plate settling, and high-rate filtration with interfilter-backwashing capabilities.

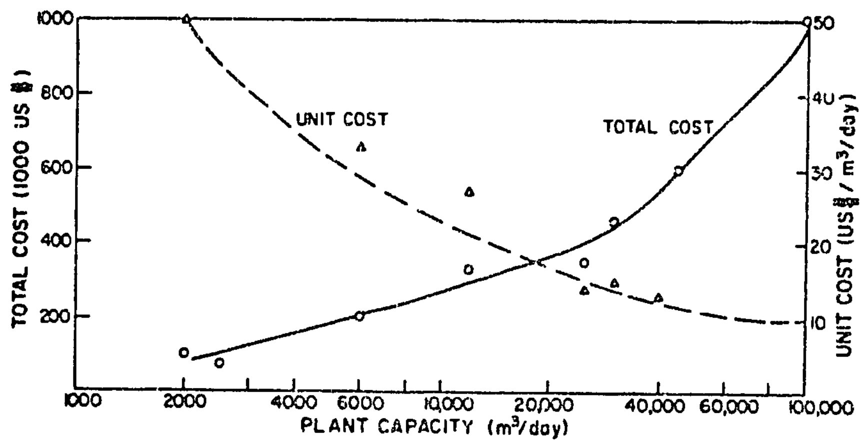
A similar Brazilian study based on package plants implemented in rural communities in the State of Sao Paulo (Azevedo-Netto, personal communication) resulted in the cost curves shown in Figure 10-3. The plants were designed with a siphon-actuated backwash system for the filters.

In India, Kardile (1981) compared actual construction costs of ten simplified upflow-downflow plants built in rural communities (see Chapter 8, "Upflow-Downflow Filtration") with cost estimates for conventionally-designed plants of the same capacities. The results are tabulated in Table 10-3. An average cost reduction of 50% results from the adoption of the simplified plants.

A detailed cost study on slow-sand filtration in India by Paramasivam, Mhaisalkar, and Berthouex (1981) gave the results presented in Tables 10-4 and 10-5; the former showing costs for filters ranging in total area from 50 to 2000 m^2, the latter showing the costs for the optimal number of filter units in a given area (assuming an increase in cost no greater than 5% to provide additional units per given filter area). From the data shown in Table 10-4, the

FIGURE 10-2

Construction Costs (1978 US$; UPC)
of Modular Plants in the
Brazilian State of Parana



[SOURCE: Richter, personal communication]

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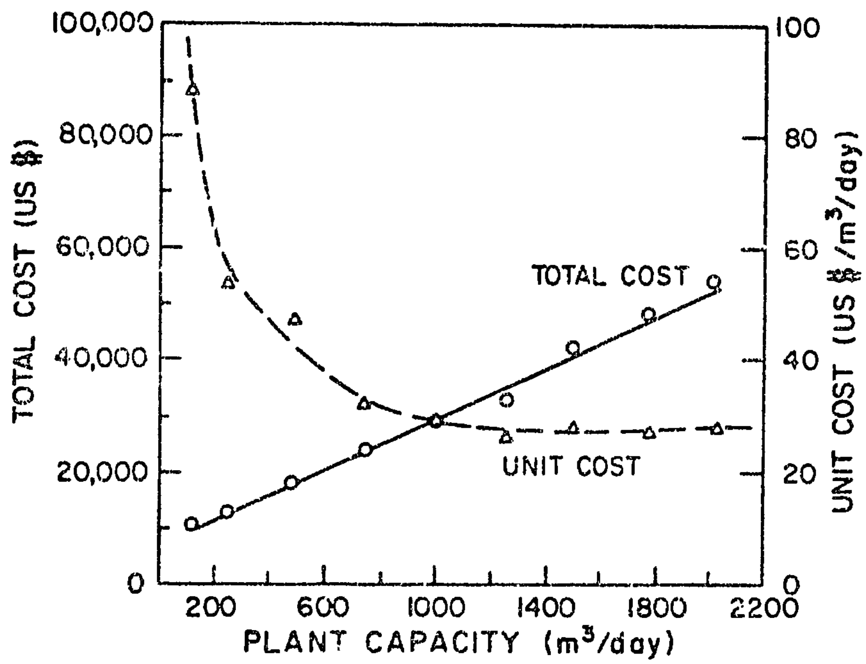
FIGURE 10-3

Construction Costs (1978 US$) of
Package Plants in the
Brazilian State of Sao Paulo



[SOURCE: Azevedo-Netto, personal communication]

TABLE 10-3: Comparative Construction costs of the Indian Upflow-Downflow Plants and Conventional Plants (*Plants described in this manual.)

LOCATION OF TREATMENT PLANT (Province)	TYPE OF PLANT	YEAR OF CONSTR.	CAPACITY (m^3/day)	CAPACITY (MGD)	TOTAL CONSTR COSTS[c] (US$)	1982 CONSTR. COSTS PER UNIT CAPACITY (US$/$m^3$/day)	1982 CONSTR. COSTS PER UNIT CAPACITY (US$/MGD)	PERCENT REDUCTION IN CONSTR. COSTS OF THE UPFLOW-DOWNFLOW PLANTS
1) Ramtek* (Nagpur)	C	1973	2400	0.64	56,000	46	170,000	
	U-D				16,000	13	49,000	71
2) Surya Colony (Thana)	C	1976	660	0.17	25,000	59	230,000	
	U-D				13,000	31	120,000	48
3) Varangaon* (Jalgaon)	C	1977	4220	1.1	100,000	34	130,000	
	U-D				50,000	17	66,000	49
4) Kandla Port Trust (Gujarat)	C	1977	2000	0.53	44,000	32	120,000	
	U-D				25,000	18	68,000	43
5) Bhagur (Nasik)	C	1978	2000	0.53	44,000	30	110,000	
	U-D				24,000	16	61,000	45
6) Murbad (Thana)	C	1978	1000	0.26	31,000	42	110,000	
	U-D				19,000	26	58,000	39
7) Jejuri (Pane)	C	1978	2400	0.63	56,000	31	120,000	
	U-D				25,000	14	53,000	56
8) Akola (Nagpur)	C	1978	2400	0.53	25,000	17	63,000	
	U-D				13,000	8.7	33,000	48
9) Dhulia dairy (Dhulia)	C	1979	1500	0.40	38,000	31	120,000	
	U-D				19,000	16	59,000	51
10) Chandori* (Nasik)	C	1980	1000	0.26	31,000	36	140,000	
	U-D				19,000	22	84,000	40

[a] C = conventional; U-D = upflow-downflow

[b] ENR Cost index for 1973 = 1895; 1976 = 2401; 1977 = 2577; 1978 = 2776; 1979 = 3003; 1982 = 3729

[c] Construction costs for conventional plants were estimated.

[SOURCE: adapted from Kardile, 1981]

TABLE 10-4: Construction Costs (1982 US$)[a] for a Given Area and Number of Slow-Sand Filter Units in India

AREA (m^2)	TWO UNITS (US$)	THREE UNITS (US$)	FOUR UNITS (US$)	FIVE UNITS (US$)
50	4,400	4,800	5,200	5,400
100	7,600	8,000	8,500	8,900
150	10,000	11,000	12,000	12,000
200	13,000	14,000	14,000	16,000
300	18,000	19,000	20,000	20,000
400	24,000	24,000	25,000	26,000
500	29,000	30,000	31,000	31,000
600	34,000	35,000	36,000	37,000
700	38,000	40,000	41,000	42,000
800	43,000	44,000	46,000	47,000
900	48,000	49,000	50,000	52,000
1000	53,000	54,000	55,000	56,000
1200	62,000	64,000	65,000	67,000
1500	76,000	78,000	79,000	82,000
2000	98,000	100,000	100,000	110,000

[a]Rs 8.8 = US$1.00
ENR Cost index for 1981 = 3533; 1982 = 3729

[SOURCE: adapted from Paramasivam, Mhaisalkar, and Berthouex, 1981, p. 180]

TABLE 10-5: Construction Cost (1982 US$) for Optimum Number of Slow-Sand Filter Units in India[a]

Area (m^2)	Number of	Cost of Filters
50	2	4,400
100	2	7,600
150	2	10,000
200	2	13,000
300	3	19,000
400	3	24,000
500	3	30,000
600	3	35,000
700	3	40,000
800	3	44,000
900	3	49,000
1000	3	54,000
1200	3	64,000
1500	4	79,000
2000	4	100,000

[a]Rs. 8.8 = US$1.00
ENR Cost Index for 1981 = 3533; 1982 = 3729

[SOURCE: adapted from Paramasivam, Mhaisalkar, and Berthouex, 1981, p. 182]

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following cost model was developed for slow sand filter beds:

$$C = 1220\ A^{0.86} \qquad (10\text{-}2)$$

where C = total construction cost (1980 rupees); A = total area of the filter beds (m^2). Hence, the cost per square meter of slow-sand filters in India in 1980 was Rs. 1220; and the exponent (b = 0.86) indicates very little economy of scale, suggesting that relatively short design periods be used in their design.

A comparative study of capital, operation, and maintenance costs for rapid and slow-sand filtration plants in India (Sundareson & Paramasivam, 1981) gave the results shown in Table 10-6 and 10-7; the former showing costs for energy, chemicals, staff, and repairs; and the latter showing total capitalized costs for both types of plants. A comparison of the total capitalized costs for rapid and slow-sand filters from Table 10-7 indicates that the costs of slow-sand filtration plants having capacities up to 23,000 m^3/day are comparable to, or even less than, those of equivalent capacity rapid filtration plants.

Operation and Maintenance Costs of Water Treatment Plants

Annual operation and maintenance (O&M) costs are highly variable among water treatment plants and much more difficult to estimate than construction costs. O&M costs depend upon labor costs, raw water quality, the extent of

TABLE 10-6: Relative Costs of Rapid Filtration and Slow-Sand Filtration in India[a]
(1982 US$ x 1,000)

Plant Capacity (m^3/day)	RAPID FILTRATION[b] Capital	Energy	Chemical	Staff Salary	Repairs and Replacement	Annual OMR Cost[a]	SLOW-SAND FILTRATION Capital	Staff Salary	Repairs and Replacement	Annual OMR Cost[d]
1,000	54	.11	2.1	4.9	1.1	8.1	38	1.5	.42	1.9
1,900	110	.11	4.1	4.9	2.1	11	66	1.5	.63	2.1
2,300	117	.11	5.0	5.7	2.3	13	79	2.2	.84	3.1
6,700	150	.32	15	6.5	6.5	24	260	2.2	2.6	4.9
15,000	340	.64	33	12	8.7	54	590	7.3	5.9	13
30,000	574	1.3	65	18	14	98	1,200	10	12	22
45,000	910	1.9	98	18	20	140	1,800	140	18	29

[a]ENR Cost Indes for 1981 = 3,533; 1982 = 3,729

[b]Includes rapid mixing, flocculation, sedimentation and filtration units

[c]Includes energy, chemical, staff, and repair charges.

[d]Includes staff and repair charges

TABLE 10-7: Capitalized Cost Estimates for Different Capacities[a] (1982 US$ x 1,000)

Plant Capacity (m^3/day)	RAPID FILTER Capital	Annual OMR	Capitalized OMR	Total Capitalized	SLOW-SAND FILTER Capital	Annual OMR	Capitalized OMR	Total Capitalized
1,000	54	8.1	62	116	38	1.9	14	52
1,900	110	11	85	195	66	2.1	16	82
2,300	120	13	99	219	79	3.1	23	102
6,700	150	24	190	340	260	4.9	37	297
15,000	340	54	410	750	590	13	100	690
30,000	570	98	740	1,310	1,200	22	170	1,370
45,000	910	140	1,000	1,910	1,800	29	220	2,000

[a] ENR Cost Index for 1981 = 3,33; 1982 = 3,729

use of imported equipment and materials, and sophistication of the facilities. Furthermore, the operating costs of a treatment plant depend to a great deal on chemical and energy costs, which are extremely sensitive to changing market prices.

Because of the highly variable nature of O&M costs, and the lack of reliable data on such costs in developing countries, cost curves for O&M are not included in the manual, although general predictive equations are presented in the next section.

O&M costs for water treatment plants are normally comprised of costs for the following elements: (1) chemicals; (2) energy; (3) personnel; and (4) maintenance materials requirements. Table 10-8 shows the variability of alum costs in developing countries. When alum has to be imported, which is the case in Nigeria, its cost is much higher.

A Brazilian cost study (Macedo and Noguti, 1978) compared the cost-effectiveness of two types of chlorine used for disinfection in water treatment; liquid chlorine and sodium hypochlorite. Costs were compared on the basis of equipment, transportation, installation, and operation and maintenance for dosages and plant capacities ranging from 1 to 5 mg/l, and 170 to 83,400 m^3/day, respectively. The component costs and total costs for a chlorine dosage of 1 mg/l are presented in Table 10-9. Sodium hypochlorite was

TABLE 10-8: Unit Costs of Alum for Several Plants in Developing Countries

City/Country	Alum (US$/ton)
Cochabamba, Bolivia	140
Linhares, Brazil	94
Prudentopolis, Brazil	110
Amman, Jordan	350
Kano, Nigeria	400
Bamako, Nigeria	700

TABLE 10-9: Comparative Costs (1982 US$) of Liquid Chlorine (40 kg and 900 kg containers) and Sodium Hypochlorite for a Chlorine Dosage of 1 mg/l - Brazil[a]

	COST COMPONENT	FLOW (m^3/day) 170	430	810	4300	8600	43,200	86,400
Sodium Hypochlorite	hypochlorite	7.8	19	39	190	390	1900	3900
	equipment	34	34	34	34	34	34	34
	transportation	21	52	100	521	1000	5200	10,000
	container	2.1	4.2	7.3	32	65	330	650
	installation	17	17	21	84	210	343	690
	TOTAL	82	130	200	860	1700	7800	15,000
Liquid Chlorine (40 kg)	chlorine	2.5	6.5	13	64	130	645	1300
	equipment	100	100	100	100	100	100	100
	transportation	.37	.98	2.0	9.7	20	98	200
	container	11	11	11	39	72	360	720
	installation	25	25	25	25	25	33	59
	TOTAL	140	140	150	240	350	1200	2400
Liquid Chlorine (900 kg)	chlorine	1.4	3.5	7.1	35	71	350	710
	equipment	400	400	400	400	400	400	400
	transportation	.27	.69	1.4	6.8	14	68	140
	container	72	72	72	72	72	140	220
	installation	84	84	84	84	84	84	84
	system for moving cylinders	110	110	110	110	110	110	110
	TOTAL	670	670	670	710	750	1200	1700

[a]Brazil Cr.$170 = US$1.00
ENR Cost Index for 1978 = 2776; for 1982 = 3729.

An interest rate of 12% has been assumed. Amortization periods of 20, 15, and 10 years have been taken for the equipment for liquid chlorine (900 kg), sodium hypochlorite, and liquid chlorine (40 kg), respectively; and one to 20 years for installation.

[SOURCE: adapted from Macedo and Noguti, 1978, p. 283] shown to be cost-effective at plant capacities below 500 m^3/day.

<u>Predictive Equations for Construction and O&M Costs</u>

Multiple regression techniques were employed by Reid and Coffey (1978) to develop predictive equations for water treatment systems in developing countries, utilizing socioeconomic, environmental, and technological indicators. Predictive equations were developed for three regions (Africa, Asia, and Latin America) for construction and O&M costs for both rapid filtration and slow-sand filtration plants. Water treatment costs were found to be a function of population, plant capacity, water demand, and the percentage of imported water supply materials. Cost equations for rapid and slow sand filtration plants are presented in Tables 10-10 and 10-11, respectively. The coefficient of correlation (R^2), included in the Tables, is indicative of how well the regression equations correlate with the set of observations (i.e. raw plant cost data) used in their formulation.

Table 10-12 shows typical construction, operation and maintenance costs of rapid and slow-sand filtration plants for selected socioeconomic and technological conditions using the predictive equations. The equations yield results in US customary units; conversions to metric units are shown in the table.

TABLE 10-10: Predictive Equations for Estimating Rapid Filtration Plant Costs in Developing Countries[a]

Region	Eq. No.	Construction Cost Equation[b]	R^2	Eq. No.	Operation & Maintenance Cost Equation[b]	R^2
Africa	10-2	$C_c = \frac{614\ I_m^{-0.10}}{P^{0.013}}$	0.86	10-6	$C_{O\&M} = \frac{197\ I_m^{0.023}}{Q^{0.037}}$	0.87
Asia	10-4	$C_c = \frac{998\ I_m^{0.007}}{Q^{0.038}}$	0.88	10-7	$C_{O\&M} = \frac{277\ I_m^{0.025}}{Q^{0.055}}$	0.90
Latin America	10-5	$C_c = \frac{777\ P^{0.003}}{Q^{0.090}}$	0.96	10-8	$C_{O\&M} = \frac{202\ I_m^{0.045}}{Q^{0.053}}$	0.97

[a]The original regression equations by Reid and Coffey were projected to 1975 US\$ assuming 6½% annual inflation. The equations shown here have been adjusted to March 1982 US\$ using the ENR cost index of 3729 and rounded off.

[b]where C_c = construction costs (1,000 US\$/MGD)

$C_{O\&M}$ = operation and maintenance costs (1,000 US\$/MGD/year)

I_m = % cost of imported water supply materials

P = design population (1,000's)

Q = plant capacity (MGD)

D_w = water demand (gallons/capita/day)

[SOURCE: adapted from Reid and Coffey, 1978]

TABLE 10-11: Predictive Equations for Estimating Slow-Sand Filtration Plant Costs in Developing Countries[b]

Region	Eq. No.	Construction Cost Equation[b]	R^2	Eq. No.	Operation & Maintenance Cost Equation[b]	R^2
Africa	10-9	$C_c = \dfrac{53.3\ D_w^{0.099}}{P^{0.132}}$	0.81	10-12	$C_{O\&M} = \dfrac{8.55}{Q^{0.620}}$	0.87
Asia	10-10	$C_c = \dfrac{62.1\ I_m^{0.010}}{Q^{0.107}}$	0.88	10-13	$C_{O\&M} = \dfrac{14.1}{P^{0.102}\ Q^{0.489}}$	0.90
Latin America	10-11	$C_c = \dfrac{75.4}{P^{0.080}}$	0.59	10-14	$C_{O\&M} = \dfrac{9.02\ I_m^{0.002}}{Q^{0.632}}$	0.58

[a] The original regression equations by Reid and Coffey were projected to 1975 US$ assuming 6½% annual inflation. The equations shown here have been adjusted to March 1982 US$ using the ENR cost index of 3729, and rounded off.

[b] where C_c = construction costs (1,000 US$/MGD)

$C_{O\&M}$ = operation and maintenance costs (1,000 US$/MGD/year)

I_m = % cost of imported water supply materials

P = design population (1,000's)

Q = plant capacity (MGD)

D_w = water demand (gallons/capita/day)

[SOURCE: adapted from Reid and Coffey, 1978]

TABLE 10-12: Estimated 1982 Costs of Water Treatment Plants Using the Predictive Equations

------- REGRESSION PARAMETERS --------

Water Demand lpcd	Water Demand gpcd	Design Population	Design Capacity m^3/d	Design Capacity MGD	1982 Construction Costs per Unit Capacity US\$/m^3/d	1982 Construction Costs per Unit Capacity US\$/MGD	1982 O&M Costs per Unit Capacity US\$/m^3/d	1982 O&M Costs per Unit Capacity US\$/MGD
RAPID SAND FILTRATION (Africa)					Eq. 10-3		Eq. 10-6	
100	26	10,000	1,000	0.26	160	610,000	58	220,000
150	40	50,000	7,500	2.0	160	600,000	53	200,000
200	53	100,000	24,000	5.3	160	590,000	51	200,000
SLOW SAND FILTRATION (AFRICA)					Eq. 10-9		Eq. 10-12	
100	26	10,000	1,000	0.26	14	54,000	5.3	20,000
150	40	50,000	7,500	2.0	13	51,000	1.5	5,600
200	53	100,000	20,000	5.3	11	43,000	0.80	3,000
RAPID SAND FILTRATION (Asia)					Eq. 10-4		Eq. 10-7	
100	26	10,000	1,000	0.26	280	1,100,000	83	320,000
150	40	50,000	7,500	2.0	260	990,000	74	280,000
200	53	100,000	20,000	5.3	250	950,000	70	270,000
SLOW SAND FILTRATION (Asia)					Eq. 10-10		Eq. 10-13	
100	26	10,000	1,000	0.26	19	73,000	5.7	22,000
150	40	50,000	7,500	2.0	16	59,000	1.8	6,700
200	53	100,000	20,000	5.3	14	53,000	1.0	3,900
RAPID SAND FILTRATION (Latin America)					Eq. 10-5		Eq. 10-8	
100	26	10,000	1,000	0.26	230	880,000	64	240,000
150	40	50,000	7,500	2.0	200	740,000	57	220,000
200	53	100,000	20,000	5.3	180	680,000	54	210,000
SLOW SAND FILTRATION (Latin America)					Eq. 10-11		Eq. 10-14	
100	26	10,000	1,000	0.26	17	63,000	5.5	21,000
150	40	50,000	7,500	2.0	15	55,000	1.5	5,800
200	53	100,000	20,000	5.3	14	52,000	84	3,200

Assumptions: cost of imported materials is 10% of total cost.

[SOURCE: Based on Reid and Coffey, 1978]

Conclusion

Of the data presented in this chapter, the relative costs among types of treatment plants (rapid, slow-sand, upflow-downflow, modular, package, direct filtration), and between simplified and sophisticated designs are useful. Two important conclusions can be drawn: (1) the design of treatment plants using simplified technologies can result in capital costs about one order of magnitude lower than those for plants designed with conventional technologies; and (2) capital and O&M costs can be lowered considerably by investigating alternative types of plants such as slow-sand filtration, direct filtration, or upflow-downflow plants in situations where they are technically feasible.

Reliable cost data on water treatment projects in developing countries are difficult to obtain, primarily because national or state agencies responsible for the planning and development of such projects generally lack the resources to evaluate and make available realistic cost data In fact, most of the cost data presented in this chapter originated in Brazil and India, which have fairly strong water supply infrastructures. In order to provide a greater diversity of cost data from other countries in future editions of this manual, individuals or organizations who have access to such data are kindly asked to submit this information to the authors at the University of North Carolina at Chapel Hill.

XI. HUMAN RESOURCES DEVELOPMENT

"Neither general programs nor even generous supplies of capital will accomplish much until the right technology, competent management, and manpower with the proper blend of skills are brought together and focused effectively on well-conceived projects." This statement, by George D. Woods, former president of the World Bank, made in an address to the UN Economic and Social Council on March 26, 1965 summarizes the dilemma that is faced in the provision of water in the developing world.

Instances are legion where even most appropriately designed water treatment facilities are properly constructed but, after a few years service, are found to be operating extremely poorly if at all. The instances where instruments are not functioning, where stocks of chemicals have been exhausted, where pump motors have burned out, where sludge cake has been allowed to accumulate and solidify in sedimentation tanks, where sand has been flushed out of filters, and where laboratory equipment and materials have not been replaced are all too common. Accordingly, a water treatment facility cannot be considered to be complete, even if the construction is finished and the equipment has been installed and operations have begun, if the personnel necessary to assure continuous maintenance of facilities are not <u>already</u> qualified and in place. Personnel requirements include not only the operators of the facility but the managers who are responsible for employing and deploying the personnel required, the technicians and craftsmen who are necessary for maintenance, and the laboratory personnel required for monitoring the operations, including the provision for their training. An ongoing training system must be in place to provide for upgrading existing personnel and to train new personnel. Furthermore, the conditions of employment and career opportunities should be such as to retain qualified personnel.

One of the major problems is that most plants are quite small and cannot afford the quality of personnel to assure their proper operation. This problem has plagued the industrialized countries as well as the developing countries. It led Britain to a regrouping of water supplies beginning in 1945 that was based upon insuring that every water supply system would be large enough to employ the services of a qualified manager, an engineer, and a chemist. The population to be served for a system to afford such personnel was estimated to be about 150,000 (Okun, 1976). The population that would be required to afford adequate personnel in the developing countries remains to be determined, as it varies substantially from place to place. However, there can be no question but that institutional development, through such devices as regional organizations, might be a device for providing the necessary qualified personnel to supervise and conduct some of the operations at the smaller facilities.

While the personnel problem constitutes one of the major constraints to proper water supply service to people in small communities in the US, the situation in the developing countries is considerably aggravated because of the inadequate institutional infrastructure for human resources development and very poor legacy in the field of general education left from the colonial period. The lead time required to develop the necessary personnel for the operation of water treatment plants in the absence of a sound basic educational resource is bound to be greater than the lead time required for designing and constructing the facilities to be operated. Nevertheless, little attention has been given to meeting this need for qualified personnel in a timely manner, either by financing institutions or by the countries themselves. Far more attention is given to the technical and financial adequacy of the project than to the personnel upon whose shoulders the ultimate success of the project must rest.

This chapter discusses the personnel requirements for the types of treatment facilities presented in this manual and something of the training required. However, the issue of human resources development for sustaining water supply projects deserves far more attention than a volume such as this can give.

One word of caution is appropriate. Because the facilities described are less sophisticated and involve less mechanical and electrical equipment than is generally found in water treatment plants in the industrialized world, this should not imply that the training may therefore be more modest. In the industrialized world, those who would be employed in water treatment plants have already had a quite good education in the public schools of the country, and they have grown up in a mechanized setting where youngsters are exposed to mechanical and electrical devices as a matter of course. The specialized training required for water treatment plant operations in such instances calls for only a relatively small amount of additional information such as might involve the chemistry of water treatment and the hydraulics appropriate to the facilities. Furthermore, should an operator be in difficulty, it is only necessary to reach for the phone to get assistance from the state agency, the purveyors of the equipment, or other experts who are within easy reach.

The operators of treatment plants in a developing country, on the other hand, have been drawn from a population with far less general education, with little mathematics and science, and will not likely have had much experience with mechnical or electrical equipment. In the event of trouble, the resources upon which to call are for assistance not available. Technical assistance is not likely to be provided by the central government and the purveyors of the equipment will be a continent away. Accordingly, the personnel responsible for water treatment facilities in developing countries must be far more self-reliant and qualified than their counterparts in the industrialized world.

A detailed examination of training needs and the strategies for meeting them, while beyond the scope of this publication, may be summarized from a report on manpower development and training in the water sector for the World Bank (Okun, 1977).

- The lack of qualified personnel constitutes a significant constraint to the successful operation of projects in the developing countries.

- If the goals of the International Decade are to be approached, hundreds of thousands of trained personnel are required to service these facilities.

- Donor agencies and the developing countries themselves, with but a few notable exceptions, have not yet given attention to human resources development nor to training at all consistent with the level of their investment in facilities, with the result that operations are generally poor. Water that should be safe is unsafe, extensions of service to unserved populations are slow, and breakdowns of service are frequent.

- Until attention to human resources development matches the priority given to technical and financial feasibility, particularly in the early stages of a project, little improvement can be expected.

- Even where a commitment to human resources development is made, little training will be undertaken unless the donors are perceived as being committed to human resour~es development themselves. This would require that donor agencies and the host countries develop institutional resources, including personnel, whose chief obligations in the water sector are to assure adequate human resources development.

- A wide spectrum of skills is required for water supply: managers, engineers, chemists and biologists, technicians and craftsmen

- The populations being served by these facilities need to be educated to the benefit of water service and safe water so they can play a role in assuring the appropriateness of the facilities to be provided for them and the quality of the operation of these facilities.

- Institutions responsible for water suppply must provide continued personnel development and career planning to avoid the attrition so common in the field, as the most qualified individuals are drawn off to other sectors or to other countries. The commitment of the water agency to human resources development, and the initiation of training programs, would not only improve the technical competence of the staff but would demonstrate to the staff that the agency has an interest in their careers.

This chapter presents the various kinds of personnel required in the various water treatment facilities, their numbers in accordance with the type and size of facilities, and the training required for such personnel.

An Overview of Manpower Development in the Developing Countries

To meet the United Nations target for the International Water Supply and Sanitation Decade (1981-1990), an additional 600 million people are to be provided with water supply. Based upon an estimated 4 employees per 1000 connections, and six people per connection, and assuming sufficient personnel for existing levels of service, this decade would require more than 400,000 additional trained personnel. The shortage of trained personnel is readily apparent in the water treatment plants of developing countries, where at least 50 percent of the installations lie idle in disrepair after construction (McGarry and Schiller, 1981). In fact, a WHO survey listing responses from 86 developing countries in 1970, as to which of eight constraints to developing water supplies was most significant, found that lack of trained personnel was rated as the second most serious constraint after insufficient internal financing (WHO, 1973).

Statistics on personnel needs in the water supply and sanitation sector in developing countries are not available. One very approximate rule of thumb is that one employee is required for each 1000 population served. Where the proportion served with sanitation facilities is much less than served with water supply, the number of employees would, of course, be less. More reliable personnel surveys and estimates have been made for individual countries, but great care is required in translating such estimates to other countries, even in the same region.

A manpower and training resources inventory was completed in 1975 (Carefoot, 1977). A summary of that inventory for five sector employers in Peru is presented in Table 11-1. Table 11-2 summarizes population and utilities employees as related to water and sewage service in Peru determined for 1975 and estimated for 1980. Based on the 1975 numbers, a manpower comparison index (ratio of utility personnel to population served) was calculated. On this basis, 6000 additional employees were needed for the water and wastewater utilities in 1980.* [*The population served with sewerage is counted twice, as it can be assumed that those with sewerage also have water service. If it is assumed that those with water service but without sewerage have some form of sanitation, then the ratio of employees

TABLE 11-1: Summary of Manpower Inventory for Water and Wastewater Utilities in Peru - December 1975

GENERAL CLASSIFICATIONS	ESAL[a]	ESAR[b]	DGOS[c]	DIS[d]	TOTAL	PERCENTAGE OF UTILITY WORK FORCE
	NUMBER OF WORKERS					
General managers and deputies	5	2	12	2	21	0.38
Advisors	7	2	2	1	12	0.22
Managers	100	21	89	31	241	4.42
Class 1	10	6	24	5	45	--
Class 2	20	8	37	10	75	--
Class 3	40	7	28	14	89	--
Class 4	30	0	0	2	32	--
Supervisors	0	10	157	9	170	3.22
Engineers	36	3	41	17	97	1.78
Other professionals	7	4	10	0	21	0.38
Architects	0	0	2	0	2	--
Accountants	0	1	6	0	7	--
Economists	1	0	0	0	1	--
Others	6	3	2	0	11	--
Technician 1	31	2	50	5	88	1.62
Technician 2	4	4	79	7	94	1.72
Qualified employees	137	26	117	28	338	6.18
Semiqualified employees	395	91	584	24	1094	20.03
Foremen	43	6	32	0	81	1.48
Qualified workers	135	0	5	3	143	2.62
Semiqualified workers	629	62	572	28	1291	23.83
Nonqualified workers	830	76	850	10	176	32.32
Subtotal	2359	309	2630	165	5463	100.00
Personnel of small communities - estimated	--	--	--	--	300	--
Personnel of rural areas - estimated	--	--	--	--	660	--
Total	--	--	--	--	6423	--

[a]Empresa de Saneamiento de Lima

[b]Empresa de Saneamiento de Arequipa

[c]Direccion General de Obras Sanitarias, Ministerio de vivienda

[d]Direccion de Ingeneria Sanitaria, Ministerio de Salud

[SOURCE: Carefoot, 1977, p. 642]

TABLE 11-2: Demographic Data for Peru Related to Water and Sewage Service

POPULATION BY CATEGORY	1975 ACTUAL	1980 ESTIMATED
Total population	15,326,000	18,527,000
Population with water service	7,289,800	12,532,000
Population with sewage service	4,777,000	11,482,460
Population with water or sewage service	12,066,800	24,014,460
Number of water and wastewater employees	6,423	12,500

[SOURCE: Carefoot, 1977, p. 642]

per 1000 population served with water supply and sewerage in 1975 was about 0.9, close to the rough estimate of 1 per 1000.] This number does not include the manpower requirements to provide for the high attrition generally experienced in developing countries.

The following conclusions were drawn from the Peru exercise:

- An estimated 80% of the 1975 work force required training to meet the demand of their jobs, representing a training backlog of about 5000.

- The top professionals in the industry represent 2.8 percent of the total labor force. Originally, particularly in Latin America, the training focus had almost exclusively been on this group.

- The total number of workers occupying semiqualified and nonqualified positions represent 76 percent of the personnel. None of the training institutions contacted offered courses for these employees.

- There is a marked shortage of appropriate manuals and teaching aids for this sector. Training manuals do not exit for some of the subprofessional job classifications.

A World Bank Sector Study of Iran (World Bank, 1975) estimated the personnel requirements for water and sewerage in that country, which are summarized in Table 11-3. In Iran with a total population of about 33,000,000 in 1975, only about 60% of the urban and 30% of the rural population

TABLE 11-3: Personnel in the Water Sector - Iran

	PROFESSIONAL	TECHNICIANS	SKILLED LABOR	TOTAL
Existing Staff - 1974				
Water	1004	3107	5680	9791
Sewerage	42	55	166	263
TOTAL	1046	3162	1846	10054
Forecast Need - 1984				
Water	1986	5695	12550	20231
Sewerage	677	1074	4172	5923
TOTAL	2663	6769	16722	26154
New Personnel Required - 1984				
Water	982	2588	6870	10440
Sewerage	635	1019	4006	5660
TOTAL	1617	3607	10876	16100

[SOURCE: World Bank, 1975, p. 6]

have adequate access to water and many fewer to adequate sanitation. The needs for personnel and their training are far greater than are likely to be met.

The development of human resources in developing countries is often characterized by the general conditions outlined below:

1) Government agencies in the least developed countries tend to employ expatriates for supervisory or technical positions on a temporary basis, with the expectation that local personnel can be adequately trained to take over these positions in the future. However, in too many instances, expensive expatriate contracts are renewed, because the expatriate has little interest in the training of individuals who would make him redundant and the expatriate becomes a permanent fixture.

2) Technically skilled and experienced personnel are difficult to attract and to retain in supervisory positions in water supply agencies because salary scales are low. Furthermore, many plants experience high rates of employee turnover because of the opportunities available for qualified personnel to work in more "prestigious" and higher paying jobs in the private sector and in other, richer, countries.

3) There is a tendency to overman water treatment plants with unskilled personnel because of high unemployment and political pressures within the country. This results in

poor performance, the underutilization of personnel, employment inflexibility, and a reluctance to learn new and improved job methods (Barker, 1976).

4) Adequate programs of preventive maintenance and a lack of spare parts characterize almost all plant operations. These are symptomatic not only of a shortage of trained operators but, more importantly, of supervisory and administrative personnel.

5) With a few notable exceptions, as for example, Brazil, Tunisia, and India, training facilities are available for only a small fraction of the professional staff now employed, with almost no provisions for those to be employed in the future. The situation for subprofessionals may be somewhat better.

6) Greater attention is now being given by financing agencies to so-called operator or technician training. This is modeled after training in the industrialized world, where such training has been perceived as being of high priority. This is the case because professional training is well institutionalized and the professionals promote the training of subprofessionals. On the other hand, the great need in the least developed countries is for professionals who can initiate the planning, design, financing, and management of projects in the water sector. It is this group that will need to be responsible for institutionalizing training for subprofessionals. Too

often, when the financing of training instituted by external financing agencies comes to an end, the training itself ends.

Classifications of Plant Personnel

The various kinds of personnel required to operate and maintain a water treatment plant are largely a function of the type and size of plant.* [*Personnel required at agency headquarters are not included here.] Table 11-4 lists the kinds of personnel and resources required for four categories of water treatment. The table shows manpower requirements to be comprised of three distinct groups: professional, skilled,and unskilled.

Professional personnel require a substantial amount of formal training, generally from a university. The superintendent of a large rapid filtration plant, for example, would fall into this group. Skilled, or subprofessional personnel require some formal training, generally a secondary school education plus two to three years of specialized vocational training. Training facilities for subprofessionals in a sector are often maintained in a developing country by the agency of the government responsible for that sector in order to meet their specific requirements for manpower. Unskilled personnel, or common laborers, require little formal training; these individuals can be provided the necessary

TABLE 11-4: Kinds of Personnel and Resources Required for Water Treatment Plants

TREATMENT METHODS	MANPOWER REQUIRED FOR OPERATION			RESOURCES REQUIRED			
	UNSKILLED	SKILLED	PROFESSIONAL	OPERATION EQUIPMENT	PROCESS MATERIALS	MAINTENANCE SUPPLIES	CHEMICAL SUPPLIES
Slow-sand filtration (conventional, upflow, dynamic)	X					X	
Conventional rapid filtration (conventional, dual-media, upflow)		X	X	X	X	X	X
Advanced rapid filtration (multi-media, plate or tube settling, polyelectrolytes)		X	X	X	X	X	X
Disinfection (chlorination)		X		X	X	X	X

[SOURCE: adapted from Reid and Coffey, 1978, p. 58]

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skills on the job, but even this requires organization and a highly qualified operating staff.

Numbers of Plant Personnel

The numbers of personnel required for the operation and maintenance of water treatment plants depend in general, on the design, layout, size, and complexity of the facility. For example, chlorinated groundwater supplies may be operated by only one person who would be responsible for checking pumps and chlorinators. Rapid filtration plants with continuous operation require a minimum of 4 operators: 1 chief operator and 3 shift operators (assisted by maintenance mechanics and laborers). For very large plants, separate groups must handle pumping stations, chemical building, filter operating floor, and laboratory; a minimum of 4 operators is needed by each group, supervised by a plant superintendent and assisted by maintenance mechanics and laborers (Cox, 1964).

Apart from the general considerations mentioned above, the following factors are important in determining manpower requirements: (1) type of supply: surface or groundwater; (2) location of the source of supply as related to location of treatment facility; (3) variability of raw water characteristics with regard to flow and quality; (4) type and complexity of equipment; (5) employment of any special treatment; and (6) requirements for pumping. The simplified designs described in this manual require a significant number of unskilled laborers periodically to handle various labor-intensive jobs such as, for example, the removal of sludge from the settling basins in a rapid filtration plant, or the removal and washing of the dirty sand from the bed of a slow sand filter.

Manpower requirements for various types of plants and population levels are summarized in Table 11-5; manpower requirements for cleaning of slow-sand filters are shown in Table 11-6 for both manual and mechanical methods; and the laboratory staff required for plants of different capacities is shown in Table 11-7.

Training

The organization of training programs and the design of appropriate curricula are beyond the scope of this manual. Many useful documents on various elements of training have been developed, and are readily available from several agencies, including the International Reference Center for Community Water supply, the US Agency for International Development, and the World Health Organization. Of particular value is the Basic Strategy Document on Human Resources Development for the Decade, published by WHO (1982), which emanated from a task force of the Decade Steering Committee representing the international, bilateral, and nongovernmental organizations in the sector, together

TABLE 11-5: Operation and Maintenance Manpower Requirements for Water Treatment Plants

TYPE OF TREATMENT	SIZE OF COMMUNITY	MANPOWER REQUIRED: UNSKILLED	SKILLED	PROFESSIONAL
Slow-Sand Filtration (conventional, upflow, dynamic)	500 - 2500	1		
	2500 - 15,000	2		
	15,000 - 50,000	5		
	50,000 - 100,000	8		
Conventional Rapid Filtration (conventional, dual-media, upflow)	500 - 2500	1	1	
	2500 - 15,000	1	1	1
	15,000 - 50,000	8	2	1
	50,000 - 100,000	10	3	1
Advanced Rapid Filtration (multi-media, plate or tube settling, polyelectrolytes)	500 - 15,000	1	1	1
	15,000 - 50,000	6	2	2
	50,000 - 100,000	10	5	2
Disinfection (chlorination)	500 - 2500	1		
	2500 - 15,000	1	1	
	15,000 - 50,000	2	1	1
	50,000 - 100,000	4	1	1

[SOURCE: adapted from Reid and Coffey, 1978, pp. 633-639]

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TABLE 11-6: Comparison of Requirements for Cleaning Slow-Sand Filters (Slow-sand filter with an area of 2000 m^2)

	MANUAL METHOD	TRACTOR SCRAPERS LONDON	TRACTOR SCRAPERS BERLIN	GENTRY SCRAPER AMSTERDAM	HYDRAULIC METHOD
Number of hours required for					
Draining	2	2	2	2	0
Cleaning	9	4	5	3	6
Refilling	5	5	5	5	0
Re-ripening	24	24	24	4	4
Total time out of service (hours)	40	35	36	14	10
Total number of men employed	8	4	2	2	1
Total number of man-hours involved	75	20	15	10	10

[SOURCE: Huisman and Wood, 1974]

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TABLE 11-7: Staff Required for Water Treatment Plant Laboratories

Size of Plant (m^3/day)	Staff (full-time)
<20,000	1/2
20,000 - 80,000	1 - 2
80,000 - 200,000	2 - 5
>200,000	>5

[SOURCE: adapted from Hudson, 1981, p. 293] with an "HRD Check List Package." Several points cannot be overemphasized:

-- The lead time required for preparing qualified personnel for plant operation is greater than for the design and construction of the plant. Hence, the initiation of a training program should enjoy a high priority in both timing and funds.

-- The organization and conduct of training programs require specialized skills, and are the role of specialists with water agencies. The grafting of training responsibilities onto an engineer with many other responsibilities is bound to result in training being inadequately served. If a training specialist is not available, an individual may be selected for the post and given the time and opportunity to prepare for training responsibilities.

-- The pedagogical approach must recognize the background of the individuals to be trained. Most will not have had much rewarding classroom experience, so formal classroom lecturing should be minimized and greater emphasis given to "hands-on" experiences. Accordingly, the training facility should be integrated with a suitable operating facility where feasible, and skilled operators, who are identified as being articulate, used in the training.

-- Before new training institutions are developed, a survey of available resources may reveal existing facilities that, with some assistance, can be adapted to the task. such facilities would include universities, teacher training institutions, vocational training schools, or other specialized establishments. Universities should be encouraged to undertake responsibilities for sub-professional training as they have much to offer in facilities, personnel, and status.

-- One important benefit of training that is often overlooked in planning is that the very fact of providing training for employees enhances the status of their positions and endows them with an aura of importance that increases interest and improves performance on the job. Associated with this view of training is the value of follow-up of trained personnel by periodic visits from managers to assure them of the importance of their work.

-- Continued education, in the several modes that are feasible, needs to be institutionalized for personnel at all levels.

-- Certification of operating personnel might be considered where appropriate.

-- Those responsible for human resources development must be adamant that training be institutionalized and that a training component be an important element of every project in the water and sanitation sector.

APPENDIX A

Common Chemicals Used in Water Treatment

[Information compiled from table prepared by B.I.F. Industries]

1. Aluminum Sulfate

CHEMICAL FORMULA	$Al_2(SO_4)_3 \cdot 14H_2O$
COMMON NAME	alum, filter alum, sulfate of aluminum
AVAILABLE FORMS	ground, rice, powder and lump form in 50- and 100-kg bags; 150 and 180-kg barrels; 10-, 50- and 125-kg drums; and carloads; available also as 50% solution.
APPEARANCES AND PROPERTIES	Light tan to gray-green; dusty, astringent, only slightly hygroscopic
WEIGHT	960 to 1200 kg/cm^3
COMMERCIAL STRENGTH	at least 17% Al_2O_3
FEEDING	fed dry in ground and rice form; maximum concentration 60 gr/l
HANDLING MATERIALS	handled dry in iron, steel and concrete; wet in lead, rubber, asphalt, cypress

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2. Calcium Oxide

CHEMICAL FORMULA	CaO
COMMON NAME	quicklime, burnt lime, chemical lime, unslaked lime
AVAILABLE FORMS	lumps, pebbles, crushed or ground in 50-kg moisture-proof bags, wooden barrels, and carloads
APPEARANCES AND PROPERTIES	white (light gray, tan); unstable, caustic, and irritating; slakes to calcium hydroxide with evolution of heat when water is added
WEIGHT	880 to 1120 kg/cm^3
COMMERCIAL STRENGTH	70 to 96% CaO
FEEDING	best fed dry as 2-cm pebbles or crushed to pass 2.5 cm ring; requires from 1.5 to 2.7 liters of water for continuous solution; final dilution should be 10%;
HANDLING MATERIALS	handled wet in iron, steel, rubber hose, and concrete; should not be stored for more than 60 days even in tight container

3. <u>Calcium Hydroxide</u>

CHEMICAL FORMULA	$Ca(OH)_2$
COMMON NAME	hydrated lime, slaked lime
AVAILABLE FORMS	powder in 20-kg bags, 50-kg barrels and carloads
APPEARANCES AND PROPERTIES	white; caustic, dusty, and irritant; must be stored in dry place
WEIGHT	560 to 800 kg/cm^3
COMMERCIAL STRENGTH	62 to 74% CaO
FEEDING	fed dry, 60 gr/l maximum; and as slurry, 110 gr/l maximum
HANDLING MATERIALS	rubber hose, iron, steel, asphalt, and concrete

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4. <u>Calcium Hypochlorite</u>

CHEMICAL FORMULA	$Ca(OCl)_2 \cdot 4H_2O$
COMMON NAME	HTH, Perchloron, Pittchlor
AVAILABLE FORMS	powder, granules, and pellets in 50-kg barrels, 2-, 7-, 50- and 140-kg cans, and 360-kg drums
APPEARANCES AND PROPERTIES	white or yellowish-white; non-hygroscopic; corrosive and odorous; must be stored dry
WEIGHT	800 to 880 kg/cm^3
COMMERCIAL STRENGTH	70% available Cl_2
FEEDING	fed as solution up to 2% strength (30 gr/l)
HANDLING MATERIALS	ceramics, glass, plastics, and rubber-lined tanks

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5. Chlorine

CHEMICAL FORMULA	Cl_2
COMMON NAME	chlorine gas, liquid chlorine
AVAILABLE FORMS	liquified gas under pressure in 50- and 70-kg steel cylinders, ton containers, cars with 15-ton containers, and tank cars of 16-, 30-, and 55-ton capacity
APPEARANCES AND PROPERTIES	greenish-yellow gas; pungent; noxious, corrosive gas heavier than air; health hazard
COMMERCIAL STRENGTH	99.8% Cl_2
FEEDING	fed as gas vaporized from liquid and as aqueous solution through gas feeder or chlorinator
HANDLING MATERIALS	dry liquid or gas handled in black iron, copper and steel; wet gas in glass, silver, hard rubber

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6. Copper Sulfate

CHEMICAL FORMULA	$CuSO_4 \cdot 5H_2O$
AVAILABLE FORMS	ground and as powder or lumps in 50-kg bags and 200-kg barrels or drums
APPEARANCES AND PROPERTIES	clear blue crystals or pale blue powder; poisonous
WEIGHT	1200 to 1440 kg/cm^3 ground; 1170 to 1280 kg/cm^3 as powder; and 960 to 1020 kg/cm^3 as lumps
COMMERCIAL STRENGTH	99% pure
FEEDING	best fed ground and as powder; maximum concentration 30 gr/l
HANDLING MATERIALS	stainless steel, asphalt, rubber, plastics, and ceramics

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7. Ferric chloride

CHEMICAL FORMULA	$FeCl_3$ (anhydrous and as solution); $FeCl_3 \cdot 6H_2O$ (crystal)
COMMON NAME	chloride of iron, ferrichlor
AVAILABLE FORMS	solution, lumps, and granules in 20- and 50-liter carboys and in tank trucks
APPEARANCE AND PROPERTIES	solution - dark brown syrup; crystals - yellow-brown lumps; anhydrous - green, black; hygroscopic, very corrosive
WEIGHT	solution weighs 1360 to 1440 kg/cm^3; crystals 960 to 1020 kg/cm^3; anhydrous chemical 1360 to 1440 kg/cm^3
COMMERCIAL STRENGTH	solution should contain 35 to 40%; crystals 60%, and anhydrous chemical 96 to 97% $FeCl_3$
FEEDING	fed as solution containing up to 45% $FeCl_3$
HANDLING PROPERTIES	rubber, glass, ceramics, and plastics

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8. Ferric Sulfate

CHEMICAL FORMULA	$Fe_2(SO_4)_3.3H_2O$; and $Fe_2(SO_4)_3.2H_2O$
COMMON NAME	Ferrifloc, Ferriclear, iron sulfate
AVAILABLE FORMS	granules in 50-kg bags, 180- and 190-kg drums, and carloads
APPEARANCES AND PROPERTIES	$2H_2O$ - red brown; $3H_2O$ - red gray hygroscopic, very corrosive, must be stored in tight containers
WEIGHT	1120 to 1150 kg/cm^3
COMMERCIAL STRENGTH	$3H_2O$ should contain 18.5% Fe; $2H_2O$ should contain 21% Fe
FEEDING	best fed dry, 170 to 290 gr/l, detention time 20 minutes
HANDLING MATERIALS	stainless steel, rubber, lead, and ceramics

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9. Ferrous Sulfate

CHEMICAL FORMULA	$FeSO_4 \cdot 7H_2O$
COMMON NAME	copperas, iron sulfate, sugar sulfate, green vitriol
AVAILABLE FORMS	granules, crystals, powder, and lumps in 50-kg bags, 180-kg barrels, and bulk
APPEARANCES AND PROPERTIES	green to brownish yellow; hygroscopic, very corrosive; store dry in tight containers
WEIGHT	1000 to 1060 kg/cm^3
COMMERCIAL STRENGTH	20% Fe
FEEDING	best fed ad dry granules, 60 gr/l; detention time 5 minutes
HANDLING MATERIALS	handled dry in iron, steel and concrete; wet in lead, rubber, iron, asphalt, cypress, and stainless steel.

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10. Sodium Carbonate

CHEMICAL FORMULA	Na_2CO_3
COMMON NAME	soda ash
AVAILABLE FORMS	crystals and powder in 50-kg bags, 50-kg barrels, 10-kg drums, and carloads
APPEARANCES AND PROPERTIES	white, alkaline, hygroscopic
WEIGHT	480 to 1040 kg/cm^3, extra light to dense
COMMERCIAL STRENGTH	58% Na_2O
FEEDING	best fed as dense crystals, 30 gr/l; detention time 10 minutes, more for higher concentration
HANDLING MATERIALS	iron, steel, and rubber hose

APPENDIX B

Hydraulic Calculations for Selected Unit Processes

This appendix contains several practical examples for the design of selected unit processes for water treatment; including:

B-1) Around-the-end (horizontal-flow) baffled channel flocculator

B-2) Gravel bed flocculator

B-3) Staircase type heliocoidal-flow flocculator

B-4) Tube-settler modules in horizontal-flow settling basins

B-5 Inclined-plate settlers in horizontal-flow settling basins

PROBLEM: Design a horizontal-flow baffled channel flocculator for a treatment plant of 2160 m^3/day capacity. The flocculation basin is to be divided into 3 sections of equal volume, each section having constant velocity gradients of 50, 35, and 25 sec^{-1}, respectively. The total flocculation time is to be 21 minutes and the water temperature is 15°C. The timber baffles have a roughing coefficient of 0.3. A common wall is shared between the flocculation and sedimentation basins; hence the length of the flocculator is fixed at 6.0 meters. A depth of 0.9 meters is considered reasonable for horizontal-flow flocculators.

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SOLUTION: (1) Design the first flocculator section with a velocity gradient of 50 s^{-1} and detention time of 7 minutes.

1. Total volume of flocculation:

$$(21/1440)\ (2160) = 32\ m^3$$

2. Total width of flocculator:

$$6.0/3 = 2.0\ m$$

3. For water at 15°C;

$$\text{viscosity } (u) = 1.14 \times 10^{-3}\ kg/m.s$$

$$p = 1000\ kg/m^3$$

(values obtained from Table 5-1)

4. Number of baffles in first flocculator section:

$$n = \left[\frac{2\mu t}{\rho(1.44+f)} \left(\frac{HLG}{Q}\right)^2\right]^{1/3} \qquad \text{(Eq. 6-4)}$$

$$= \frac{2\ (1.14\times10^{-3})\ (7)\ (60)}{1000\ (1.44 + 0.3)} \left(\frac{0.9\ (6.0)\ (50)}{2160/86{,}400}\right)^2 \ 1/3$$

$$= 42$$

5. Spacing between baffles:

$$6.0/42 = 0.14m$$

minimum channel spacing is fixed by the design at 0.15 m.

6. Number of baffles based on channel spacing of 0.15 m:

$$6.0/0.15 = 40$$

7. Head loss in the flocculator section:

$$H = \frac{\mu t}{\rho g} \; G^2 \qquad \text{(adapted from Eq. 6-1)}$$

$$= \frac{(1.14\text{x}10^{-3})\ (7)\ (60)}{1000\ (9.8)} \quad (50)^2$$

$$= 0.12 \text{ m}$$

8. The same series of calculations is repeated for the remaining two flocculator sections. The results are as follows:

2nd Section - G = 35 sec^{-1}
- t = 7 minutes
- number of baffles = 33
- spacing between baffles = 0.18 m
- head loss = 0.06 m

3rd Section - G = 25 sec^{-1}
- t = 7 minutes
- number of baffles = 26
- spacing between baffles = 0.23 m
- head loss = 0.03 m

a) The total head loss in the flocculator:

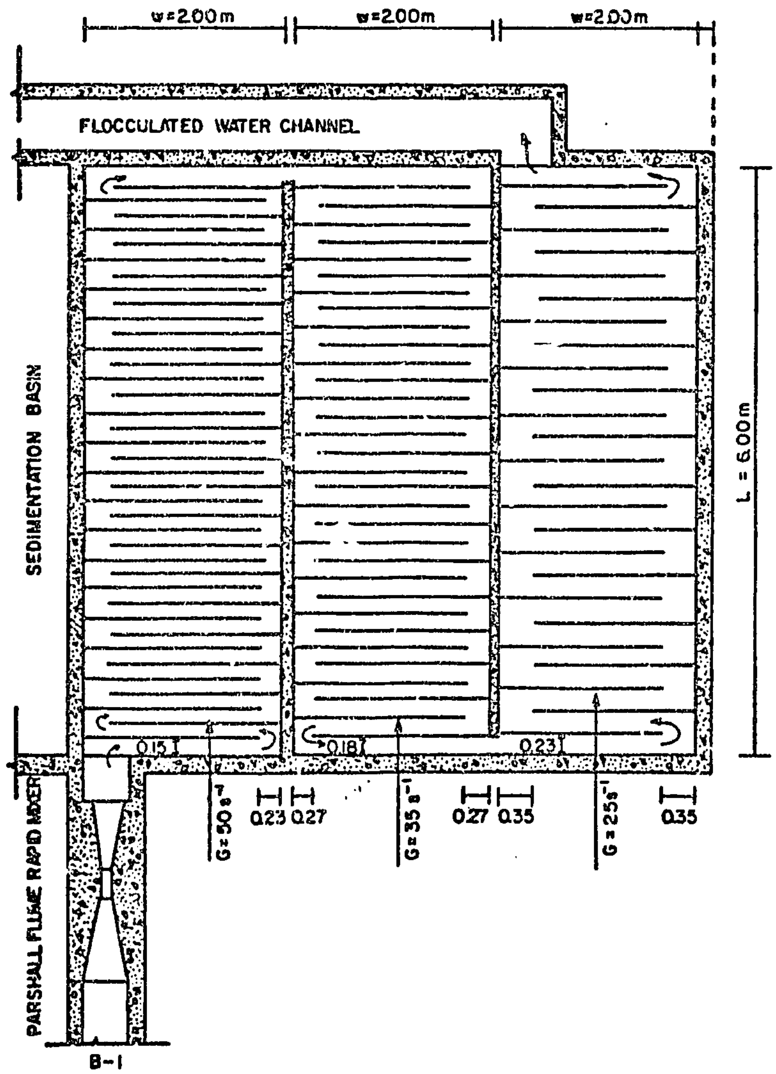
$$H = 0.12 + 0.06 \div 0.03 = 21 \text{ m}$$

The design of the horizontal-flow baffled channel flocculator is shown in Figure B-1.

* * * * *

FIGURE B-1

Horizontal-Flow Baffled Channel Flocculator for a Plant of 2160 m^3/day Capacity



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B-2 Staircase-type Heliocoidal-flow Flocculator

PROBLEM: A plant having capacity of 12,960 m^3/day contains a flocculator comprised of four square chambers with 25 minutes detention time. For one of the chambers design a staircase-type flocculator with a velocity gradient of 40 sec^{-1}. The water temperature is 20^oC and the chamber depth is 3.5 m.

SOLUTION:

1. Volume of chamber:

$$(12,960)(25)/(4)(1440) = 56.3 \ m^3$$

2. Cross-sectional area of chamber:

$$56.3/3.5 = 16.1 \ m^2$$

3. Length of one side of chamber:

$$(16.1)^{1/2} = 4.01 \ m$$

4. For water at 20^oC:

$$p = 998 \ kg/m^3 \qquad u = 1.01 \times 10^{-3}$$

(values obtained from Table 5-1)

5. Rearrange Equation 6-6 to solve for head loss:

$$h_1 = [\frac{2pKQ^3}{L^4G^2}]^{1/3}$$

$$= [\frac{2\ (998)\ (7.5)\ (12960/86400)^3}{1.01x10^{-3})\ (4.01)^4\ (40)^2}]^{1/3}$$

$$= 0.49 \ m, \ 0.5 \ m$$

6. Number of helices in the flocculator:

$$3.5/0.5 = 7.0$$

c/44

7. Value of pitch:

$$3.5/7.0 = 0.5 \text{ m}$$

* * * * *

B-3 Gravel-bed Flocculator

PROBLEM: A package water treatment plant having a capacity of 270 m^3/day contains a gravel-bed flocculator that is comprised of five rectangular sections, each of which is succeedingly larger in cross-sectional area, as shown in Figure 6-14. The dimensions of the flocculator sections and corresponding gravel sizes are given below. The gravel has a porosity of 0.4, and the water has a specific gravity and dynamic viscosity of 1.0 gr/cm^3 and 0.01 gr/cm x sec, respectively.

Section	Length (cm)	Width (cm)	Heighth (cm)	Gravel Size (cm)
1	100	5.3	20	0.5 to 1
2	100	12.6	20	0.5 to 1
3	100	23.3	20	0.5 to 1
4	100	35.4	20	1 to 2
5	100	50.5	20	1 to 2

Calculate the nominal flocculation time in the system and the velocity gradients and head loss for each section.

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SOLUTION: (1) Nominal flocculation time.

1. Volume of flocculator:

$$100\ (20)\ (5.3+12.6+23.3+35.4+50.5) = 254{,}200\ \text{cm}^3$$

2. Conversion of flow rate:

$$270(1{,}000{,}000/86{,}400) = 3125\ \text{cm}^3/\text{sec}$$

3. Nominal flocculation time:

$$254{,}200/3125 = 81\ \text{sec}$$

(2) Head loss and velocity gradient for Section 1

1. Calculate the coefficients a and b in the head loss equation:

$$a = \frac{0.162\ (1-0.4)^2\ (0.01)}{(0.8)^2\ (0.75)\ (0.4)^3} = 0.03 \qquad \text{(Eq. 6-9)}$$

$$b = \frac{0.018\ (1-0.4)}{(0.8)\ (0.75)\ (0.4)^3} = 0.28 \qquad \text{(Eq. 6-10)}$$

2. Face velocity:

$$v = 3125/100(5.3) = 5.9\ \text{cm/sec}$$

3. Volume:

$$V = 100(5.3)(20) = 10{,}600\ \text{cm}^3$$

4. Head loss:

$$h_1 = 0.03(5.9) + (0.28)(5.9)^2 = 9.9\ \text{cm} \qquad \text{(Eq. 6-8)}$$

5. Velocity gradient:

$$G = \left[\frac{9.9\ (1.0)\ (980)\ (3125)}{(0.01)\ (0.4)\ (10{,}600)}\right]^{1/2} \qquad \text{(Eq. 6-7)}$$

$$= 846\ \text{sec}^{-1}$$

(3) The same series of calculations are repeated for the remaining four sections of the flocculator. The results are as follows:

SECTION 2: h_1 = 1.8 cm G = 234 sec^{-1}

SECTION 3: h_1 = 0.54 cm G = 94 sec^{-1}

SECTION 4: h_1 = 0.24 cm G = 51 sec^{-1}

SECTION 5: h_1 = 0.13 cm G = 31 sec^{-1}

* * * * *

B-4 <u>Tube-settler Modules in Horizontal-flow Settling Basins</u>

PROBLEM: A water treatment plant, having a capacity of 114,000 m^3/day, includes two horizontal-flow settling basins, each of which is 24.4 m long, 18.3 m wide, and 3.7 m deep. Calculate (1) the actual surface loading rate (settling velocity) of the basins; and (2) the surface loading rate (settling velocity) that would be obtained if prefabricated modules comprised of square tubes inclined at 60^o are installed the last 12.5 m of the basin. The modules are 61 cm high and the cross-sectional area of each tube is 5.1x5.1 cm.

SOLUTION: <u>(1) Surface loading rate for each basin without tube settlers</u>

1. Surface loading rate:

$$S_c = 114{,}000/(18.3)(24.4)(2) = 128 \text{ m/day}$$

<u>(2) Surface loading rate for each basin with tube settlers installed</u>

1. Coefficient of performance for square tube settling system:

$$S_c = 1.38$$

(NOTE: S_c = 1.33 for circular tubes;
= 1.0 for parallel plates)

2. Relative settler length:

$$L_R = L/d = 61/5.1 = 12.0$$

3. Area of high-rate settling:

$$A = (12.5)(18.3) = 229 \text{ m}^2$$

4. Average flow velocity for area of high-rate settling:

$$v_o = 114,000/(229)(2) = 249 \text{ m/day}$$

5. Surface loading rate of tube-settlers:

$$S_o = \frac{S_c\ v_o}{\sin\theta + L_R \cos\theta}$$

$$= \frac{(1.38)\ (249)}{0.866 + (12)\ (0.5)}$$

$$= 50 \text{ m/day}$$

* * * * *

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B-5 Inclined-plate Settlers in Horizontal-flow Settling Basins

PROBLEM: The settling capacity of a water treatment plant is to be increased from 19,000 m^3/day to 48,400 m^3/day. There are three horizontal-flow settling basins, each of which is 23.5 m long, 12.0 m wide, and 4 m deep. Parallel plates are to be placed 5 cm apart at an angle of 60^o from the horizontal. The plates are 2.4 m long, 1.0 m wide, and 1.0 cm thick. The water is being treated mainly for color removal, hence the surface loading rate should not exceed 30 m/day. Calculate the area required for high-rate settling.

SOLUTION:

1. Relative settler length:

$$L_R = 100/5.0 = 20$$

2. Total area required for high-rate settling:

$$A = \frac{Q\ S_c}{S_o(\sin\theta + L_R \cos\theta)} = \frac{(48,400)\ (1)}{30[0.86 + (20)\ (0.5)]} = 150\ m^2$$

3. Area required per basin

$$150/3 = 50\ m^2;\ \text{or}\ 12\ m \times 4.2\ m$$

4. Number of plates needed:

4.2/0.05 = 84 plates per row of 2.4 m width

5. Total length of basin that will be covered by the plates:

$$0.84 + 4.2 = 5.04\ m$$

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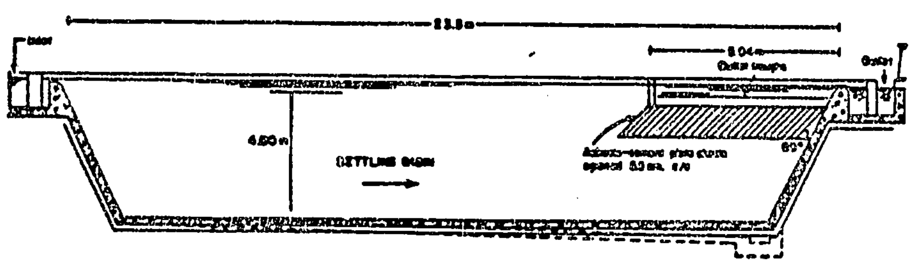
A diagram that shows the installation of parallel plate settlers in the horizontal flow basin is presented in Figure B-2.

FIGURE B-2

Installation of Inclined-Plate Settlers in a Horizontal-Flow Settling Basin



4.50

APPENDIX C

Check List for Design of Water Treatment Processes
(adapted from Hardenburgh and Rodie, 1961)

CHECK LIST FOR DESIGN OF WATER-TREATMENT PLANTS

The purpose of this list is to assemble in an orderly manner the various items important in a water treatment plant designed to treat surface water. This permits utilizing the list as means of ensuring that essential points have not been overlooked, either in a preliminary or final design.

In general, the check list is limited to the more commonly used processes. An attempt has been made to separate the items into functional and operational considerations, as far as these apply. Items such as intakes and pumping stations, not included in this manual, are covered for completeness.

PLANT AND BUILDING DATA

Plant Site: Distance from city....., access roads....., rail siding....., ground elevation....., protection against flooding....., size of property....., fencing....., landscaping....., outdoor lighting....., provision for future expansion......

Building: Type of structure....., size....., exterior finish...... **Chemical storage:** lime....., alum....., iron....., salt....., other....., unloading and handling

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methods...... **Facilities:** drinking water....., toilet....., locker room....., washroom and shower....., lunchroom....., toolroom....., shop...... **Laboratory:** **Bacteriological:** refrigerator....., incubator....., oven....., microscope....., balance....., still...... **Chemical:** hood....., pH meter....., colorimeter....., residual chlorine....., reagents...... **General:** glassware....., sinks....., hot water....., vacuum....., electricity....., lighting....., gas....., air....., safety shower....., fire protection...... **Material storage:** existing....., needed....., provided....., how......

WATER SUPPLY

Source: Surface water...... **Expected yield of source:** average....., minimum.

Population Served: Present....., design...... **Water Use:** present.....m^3/day, design.....m^3/day, average per day....., maximum month....., maximum day......

Stream Flow: Average.....m^3/day, maximum.....m^3/day, minimum.....m^3/day, high water level....., low water level......

Reservoir: Area....., depth....., HWL....., LWL......

Range of Raw Water Quality: MPN....., pH....., total solids....., turbidity....., temperature....., color....., taste and odor....., alkalinity....., hardness....., algae......

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RAW WATER TRANSMISSION

Supply Line: Number....., size....., material....., length....., delivery......m^3/day, C =....., gravity....., pumping....., pressure at plant....., head pumped against....., velocity in line for design flow....., corrosion protection....., interconnections....., air relief valves....., drains at low points....., isolation of sections for repairs....., access to right of way....., is line metered?.....

Pumping Stations: Location....., number of pumps....., capacity of pumps....., size of suction lines....., size of discharge lines....., type of pumps....., efficiency....., motive power....., power requirements....., flood protection.....

INTAKES AND SCREENS

Intakes: Number....., type....., size....., capacity....., head loss....., invert elevation of pipe out....., **elevation water surface:** high....., low....., average....., depth of water....., distance intake from shore.....

Screens: Where used....., what kind....., material....., mesh or opening size....., power requirements....., area of openings....., flow through screen.....m^3/day, are screens removable?....., are there duplicates?....., method of cleaning.....

COAGULATION AND SEDIMENTATION

Chemicals: Kinds used....., design dosages.....

Rapid Mix: Number of tanks....., tank length....., width....., depth....., retention....., type of mixer....., point of chemical feed.....

Flocculator: Number of tanks....., tank length....., width....., depth....., retention....., type of mixer.....

Ports or Openings: Rapid mix to flocculator....., velocity....., flocculator to sedimentation....., velocity....., weir or baffle adjustment possible....., can tanks be drained?....., are walkways and guard rails provided?.....

Feeders: **Dry:** Number....., capacity....., **Liquid:** number....., capacity.....

Settling Tanks: Number....., length....., width....., depth....., diameter....., retention time....., overflow rate....., flow line elevation....., sludge removal....., effluent discharge....., type of weirs....., weir overflow rate....., effluent pipe to....., tank freeboard....., can tanks be drained?....., where to?....., are walkways and guard rails provided?.....

FILTERS

Units: Size....., number of units....., area....., rate of filtration....., are walkways and guard rails provided?.....

Filter Media: **Fine medium:** material....., effective size....., uniformity....., depth...... **Coarse medium:** material....., sizes.......

Underdrainage: type.....

Backwash: Rate....., water required....., where from....., wash water trough spacing....., trough size....., shape....., slope....., lip elevation above filter surface...... **Pump:** size....., capacity.....

Washwater Tank: Capacity....., elevation above filter....., size of outlet pipe....., method of filling....., delivery to each filter.....m^3/m^2, total per minute to each filter.....

Filter Controls: Type....., location....., sewer to where....., cross-connection possible....., surface wash provided....., nozzle velocity.....

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CHLORINATION

Chlorinators: Number....., type....., capacity....., where located....., point of chlorine application....., design dosage....., contact period provided....., are chlorinators in separate building?....., is chlorine room isolated?....., gas withdrawal rate....., scales.....

Chlorine containers: Size....., storage for containers....., equipment for handling containers.....

Safety precautions: Equipment provided....., adequate exhaust system....., louvers in door....., inside fixed window....., door opening outward....., light switch near door.....

PLANT STORAGE AND PUMPING

Storage: Clear well: location....., capacity.....
Other low-level storage at plant: type....., capacity.....

Pumping: Where to?....., number of pumps....., sizes....., type....., drive....., controls....., standby power....., standby pump provided....., capacity....., power source....., disconnect switch for each pump?.....

APPENDIX D

Simple Methods for Water Quality Analyses

Test Methods

1. pH

   A. Use a square bottle, take 15 ml of water sample. Add 1 drop of solution and 1 drop of phenolphthalein indicator solution.

   B. Observe closely the color of solution.

Table 1

COLOR OF SOLUTION	pH RANGE	COMMENT ON pH
Yellow	<6.0	too low
Blue	6.0 - 8.5	OK
Purplish-blue	8.5 - 9.5	Still OK
Red	>9.0	Too high

   C. Record result in data sheet.

   D. If having difficulty in identifying the red color, the following standard red solution may be prepared: Repeat step A, but in addition, add 5 drops of NaOH solution. This should give a standard red color solution for comparison.

2. Turbidity

   A. Fill the sample bottle completely full with water sample.

   B. Shake the standards by inverting them.

   C. Compare the sample with the two standards and determine if the sample is less than 25 JTU, between 25-50 JTU, or greater than 50 JTU. (Observe movement of particles in solution).

   D. Record result in data sheet.

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3. Chlorine Residual

   A. Fill a clean bottle with water sample up to the bottle-neck. (If water sample is turbid with color, fill a second bottle as with the first bottle. This is for comparison of color in later steps.)

   B. Let the water sample(s) stand for ten minutes.

   C. Add two crystals of potassium iodade. (DO NOT ADD THIS TO THE SECOND BOTTLE.)

   D. Add five drops of starch solution (TO BOTH BOTTLES).

   E. Shake the sample(s) vigorously and let it stand for five minutes.

   F. Observe the solution for change of color. Any change of color intensity upon longer standing should be disregarded.

   G. Record result in data sheet. No color - absence of chlorine (0 ppm); Faint blue color - correct amount of chlorine (0.15 ppm); Dark blue color - too much (>0.2 ppm).

4. Coliform

   A. To get better results, water sample should be thoroughly swirled before use.

   B. To the first group of five bottles (with correct amount of media and indicator solution), introduce 10 ml of water sample into each, by using the syringe. <u>BE SURE</u> to record the amount of water sample introduced into each bottle.

   C. To the second group of five bottles, introduce 1 ml of water sample into each.

   D. To the third group of five, introduce 0.1 ml of water sample into each.

   E. Incubate the bottles at 35C (or 95F) for 48 hours.

   F. After 48 hours, observe for color change in the bottles. Bottles that have changed from purple to yellow color indicate a positive test. Record the number of bottles in each concentration that give positive results.

G. MPN Index (most probable number) and most positive results. Domestic water supply - see Table 2.

H. Record result in data sheet. (If MPN is greater than 2 for domestic water supply rerun the test the next day or sooner, if possible.)

Table 2

MPN Index for Various Combination of Positive Results (for Domestic Water Supply)

NUMBER OF BOTTLES GIVING POSITIVE RESULTS			
10 ml Water sample	1 ml Water sample	0.1 ml Water sample	MPN Index per 100 ml
0	0	0	<2
0	0	1	2
0	1	0	2
1	0	0	2
1	0	1	>2
1	1	0	>2

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CHECKLIST BEFORE GOING TO FIELD

1. pH	A.	Bromcresol purple and phenolphthalein indicator solutions.
	B.	One clean bottle.
2. Turbidity	A.	25 JTU and 50 JTU standards* (be sure they are securely capped and not leaking).
	B.	One clean bottle (same as those containing the standards).
3. Chlorine Residual	A.	Potassium iodide crystals.
	B.	Starch solution* in dropper bottle
	C.	Two clean bottles
4. Coliform	A.	Fifteen sterilized screw-capped bottles, each containing 15 ml of media* and three drops of bromcresol purple indicator solution*.
	B.	One clean 10 ml syringe
	C.	One clean 100 ml beaker for sample collection
7. Temperature	A.	Thermometer

----

*See the section about preparation of reagents.

PREPARATION OF REAGENTS

Important: Label all containers that have reagents in them!

1. pH

A. To prepare bromcresol purple indicator solution: Dissolve 2 spoons* (use spoon B, about 0.05 gm) of bromcresol purple indicator in the dropper bottle with distilled water and fill the bottle to the neck.

B. To prepare phenolphthalein indicator solution: Dissolve 6 spoons* (use spoon B) of phenolphthalein indicator in the dropper bottle with 50 to 60% alcohol and fill the bottle to the neck.

2. Turbidity

A. To prepare stock solution: Add 1 spoon* (use spoon B) of Fuller's Earth to 50 ml of distilled water. This makes a stock solution with a turbidity of 1000 JTU.

B. To prepare 50 JTU solution: Shake the stock solution well. Take 5 ml of stock solution and dilute to 100 ml with distilled water. This makes the 50 JTU solution standard.

C. Preservation: Add mercuric chloride (a few specks) or bleach (a few drops) to each standard solution. Standards must be prepared fresh each month.

D. LABEL ALL SOLUTIONS PREPARED.

3. Chlorine residual

To prepare starch solution:

Measure out one spoon of clean starch with spoon A. Add enough cold water and stir to produce a thin paste. Add approximately 100 ml of boiling water and keep stirring. Boil for 2 to 3 minutes. Add a few drops of chloroform (or formaldehyde) to preserve the solution. Fresh solution should be prepared as often as possible (two weeks or less).

4. Coliform

A. To prepare media: Any of the following four methods may be used:

i. Rice Broth: Boil 25 grams (or fill 1 square bottle full) of rice and add 4 spoons (use spoon A, about 1 gram) of powdered milk in 450 ml of water for 5 minutes, stir occasionally. Decant carefully the rice broth into a glass bottle and discard the rice residue.

ii. Potato Broth: Peeled or sliced potatoes (or sweet potatoes) may be used. Boil 50 grams of potato (in place of the 25 grams of rice) and 4 spoons (spoon A) of powdered milk for 15 min, then follow the same steps as with the rice broth.

iii. Corn Meal Broth: Heat 400 ml of water to 70C (158F). Add 1 square bottle full of corn meal and 4 spoons (use spoon A) of powdered milk, stir frequently. Decant carefully the broth into a glass bottle and discard the residue.

iv. Lactose Broth: Dissolve 1/4 of a beef bullion bar (approximately 1 gram) and 4 spoons (use spoon A) of powdered milk in 250 ml of distilled water. Heat if necessary.

B. To prepare sterilized culture bottles: Take 15 clean, screw-capped bottles. Introduce 15 ml of media into each bottle. Add 5 drops of bromcresol purple indicator solution to each bottle. Sterilize with the cap loosely placed on the mouth of the bottle. Let cool slightly; tighten the cap.

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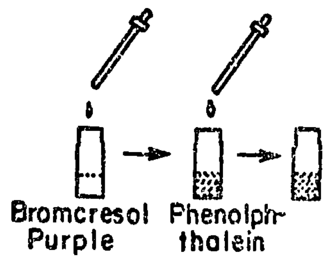
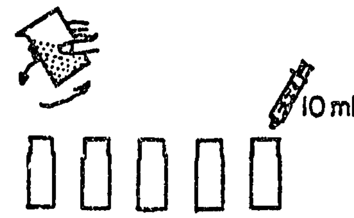
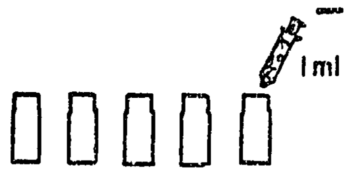
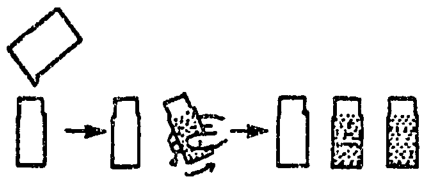
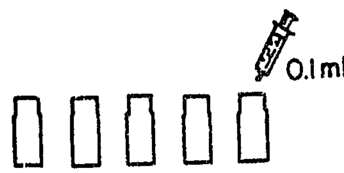
*One spoon of reagent: Fill the spoon with one level spoonful of reagent, use a sheet of paper to scrap off the excess from the top and the sides. Invert the spoon, tap the end of the spoon handle to release the powder.

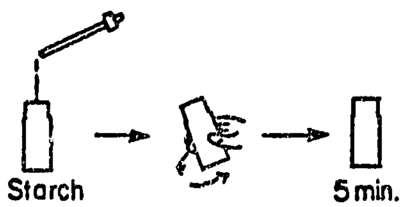

Bromcresol
Purple
Phenolph-
thalein

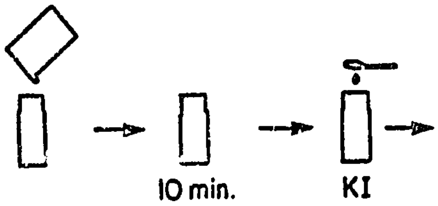

10 ml

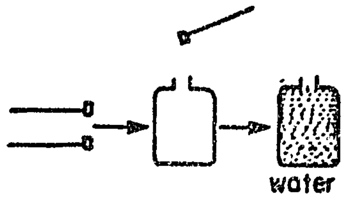

1 ml

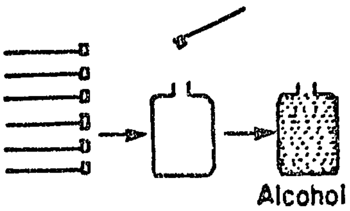


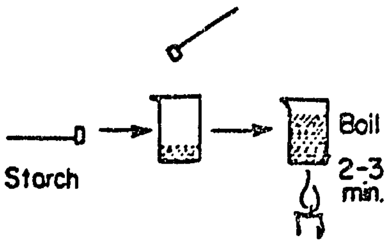

0.1 ml

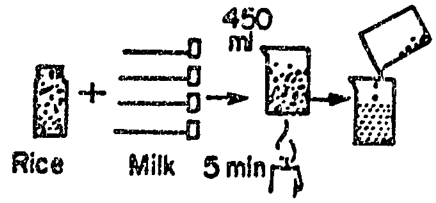

Starch
5 min.

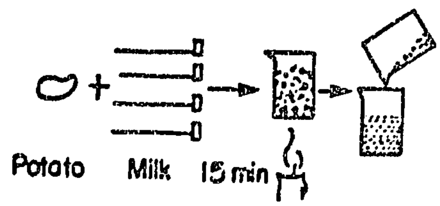

10 min.
KI

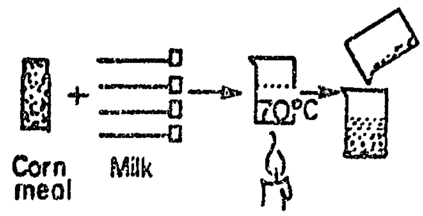

water

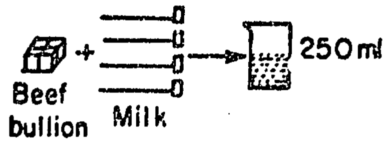

Alcohol


Starch
Boil
2-3
min.


450
ml
Rice
Milk
5 min


Potato
Milk
15 min


Corn
meal
Milk
70°C


Beef
bullion
Milk
250 ml

APPENDIX E

Glossary of Organizations

AWWA	American Water Works Association 6666 West Quincy Avenue Denver, Colorado 80235 USA
AIT	Asian Institute of Technology PO Box 2754 Bangkok, Thailand
CEPIS	Pan American Sanitary Engineering and Environmental Sciences Center (CEPIS) Casilla 4337 Lima, 100, Peru
ENSIC	Environmental Sanitation Information Center Asian Institute of Technology PO Box 2754 Bangkok, Thailand
GTZ	German Agency for Technical Cooperation Postfach 5180 D-6236 Eschborul, West Germany
IBRD	International Bank for Reconstruction and Development World Bank 1818 H Street NW Washington, DC 20433 USA
IDRC	International Development Research Center PO Box 8500 Ottawa, Canada K1G 3H9
IRC	International Reference Center PO Box 5500 2280 HM Rijswijk The Netherlands
NEERI	National Environmental Engineering Research Institute Nehru Marg Nagpur - 440 020, India
PAHO	Pan American Health Association 525 23rd Street NW, Room 523 Washington, DC 20037 USA

UNC	International Programs Library Department of Environmental Sciences and Engineering University of North Carolina Chapel Hill, North Carolina 27514 USA
WASH	1611 North Kent Street Suite 1002 Arlington, Virginia 22209 USA
WHO	World Health Organization Environmental Health Technology and Support Division - GWS 1211 Geneva 27, Switzerland

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