GUIDES TO THE DESIGN

OF

WATER TREATMENT PLANTS

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PREFACE

Criteria as to design practice warrant careful study to establish those phases which have become accepted as normal and those phases which require adaptation to local conditions and availability of materials and equipment, thus giving the designing engineer freedom of choice and the exercise of his professional judgment.

The normal or standard should not be sought merely to achieve uniformity, because that might freeze practice and stifle advance. Therefore, the criteria discussed herein are aimed to distinguish good practice by establishing quantitative limits found by experience to be desirable so as to simplify practice and thereby achieve economy.

This text has been based upon the "Report of Committee of the Great Lakes - Upper Mississippi River Board of State Sanitary Engineers on Policies for the Review and Approval of Plans and Specifications for Public Water Supplies." Modifications and condensations have been made by the author, who was chairman of the above committee, to meet the needs in Brazil and Jamaica, as disclosed by his participation in the cooperative sanitary engineering programmes in these countries between the two governments and the International Cooperation Administration of the United States.

The appendix entitled, "A Standardized Design for a Small Water Filtration Plant" was prepared in Brazil where metric units are used. Inasmuch as the plans are in these units, the text has not been changed, but conversion factors have been given.

CHAPTER I. INTRODUCTION

The design of water works involves the consideration of many engineering details in the light of specific local conditions. Details of design, therefore, cannot be standardized in any fixed pattern which would eliminate the necessity for the engineer to exercise professional judgment in appraising local conditions and in determining what structures and equipment will best meet these conditions. This does not mean, however, that each engineer can function independently, without due regard to opinions, attitudes and practices of other engineers. There is need, therefore, for group action to blend different viewpoints and attitudes so as to eliminate extremes and idiosyncrasies. Such group action is most productive when it is directed to the formulation of standards of professional practice.

There is considerable confusion in the use of the work "standard". It is desirable, therefore, to discuss several aspects of the subject, so as to distinguish between:

a) detailed specifications;
b) standard specifications;
c) standard as to best engineering practice; and
d) governmental laws, codes, and requirements.

Specifications accompanying plans for a single project are of necessity detailed, specific and binding upon those contracting to supply equipment and supplies. They express the intent of the designer and are the result of detailed consideration of proposed structures, facilities and equipment by the designing engineer, which must meet with the approval of the owner of the intended works before bids are accepted. On the other hand, "Standards", such as those developed by committees of the American Water Works Association, are intended to secure uniformity of practice, insofar as possible, but they are not binding on the individual using them. In other words, they serve as a guide in the preparation of detailed specifications for a single project and, therefore, can be used in whole or in part at the discretion of the engineer.

By contrast, statements as to best engineering practice, or features of basic design, developed by groups of engineers or committees of professional societies, are not worded as specifications but rather as guiding texts which summarize the consensus of the group in a manner intended to secure reasonable uniformity of practice. Such standards, therefore, are basic and quantitative only where there is agreement as to details which can be expressed by definite values, but merely summarizing when they deal with details which cannot be delimited by quantitative terms.

On the other hand, governmental laws, codes or requirements should be worded in a general way so as to indicate those aspects or features of design which can be insisted upon in the public interest under the police power of the state supporting the program of the governmental agency concerned with such matters. Governmental regulations, therefore, should represent minimum requirements of the agency, which, however, can be exceeded at the volition of the designing engineers.

The following text deals with basic criteria of water treatment plant design and thus falls into the third category. In general the text deals with established practice with moderate sized plants.

Imperial gallons are used, except in Chapter XIV where metric units are used to describe an actual design of a small filtration plant in Brazil.

CHAPTER II. GENERAL

Engineering Reports - Plans and Specifications

Objectives - The design of water works for a specific municipality should be based upon local conditions and requirements, with the objectives of the best usage of water resources of the locality. Good engineering and public interest require that economy in construction costs be the aim, consistant with durability and reasonable cost of maintenance. This is usually secured with simplicity of design and by avoiding, insofar as possible, mechanical equipment, automatic controls and complex units which are difficult and costly to maintain and repair, especially when the equipment and parts must be imported.

Careful consideration should be given to the number, training and experience of operating personnel who will be in charge of the system, so there will be assurance that they will be able to provide effective control and operation of the system. In many instances available operating personnel will have little or no training and experience, in which case complex water treatment processes and equipment should not be used, but stress should be placed upon the development of well supplies to minimize treatment, or consideration should be given to the use of slow sand filters for the clarification of a surface water supply, which of necessity must be used. Supplies flowing by gravity also are most desirable.

Therefore, engineering planning should include a thorough investigation of the quality and characteristics of available sources of water supply, to determine the most economical source which can be used either without treatment or with simple treatment, consistent with the required quality of the water to be served the public.

Consideration also should be given in the design to the cost of operation and maintenance of the system to insure that these costs will be within anticipated revenue or budget. There will be many designs which will have to be altered or simplified in the interest of economy. This may involve technical compromises which should be clarified in the engineering report.

The anticipated growth in population and increase in water consumption are difficult to predict. Both are influenced by considerations such as industrial development and improved transportation. Accordingly, plans should be made for future growth and expansion of water supply facilities. Reserve capacity should be provided initially for anticipated needs of the near future. The initial design also should be such that future extensions can be made with economy and so as to provide an enlarged system which will function well. This is especially desirable with pumping stations and treatment plants. The period for which anticipated needs should be projected cannot be rigorously stated; but it is reasonable to state that a water-supply should be designed to serve no less than the population expected ten years from the design date, and, if feasible, one generation thence.

CHAPTER III. AERATION

Objectives - Aeration of water is practied for three purposes, namely: 1) the release of taste and odor producing gases such as hydrogen sulphide and other volatile substance; 2) the release of carbon dioxide; and, 3) the absorption of oxygen from the air to oxidize iron and manganese. Hydrogen sulphide is readily dissipated by limited aeration, but the carbon dioxide content cannot be reduced below about 4.5 ppm. Dissolved oxygen increases the corrosiveness of water, so its concentration should not be increased by aeration except when oxygen is needed to oxidize iron or manganese, or when the increase must be accepted to gain other advantages of aeration.

Aeration is achieved mainly by the following devices:

Spray aerators - Spray nozzles provide the most complete exposure of water surface to the atmosphere, the surface area depending upon the size of the water drops, a large number of small droplets having the largest area per unit of volume. Water pressures should be 5 to 10 psi. at the throat of the nozzle. The spacing of the nozzles depends upon their capacity, which usually ranges from 32 to 140 gpm., requiring from 70 to 400 sq. ft. of area per nozzle. The spray should fall into a shallow, concrete basin.

Spray aerators should be surrounded by a louvered fence to prevent spray being blown outside the underlying basin, the louvers sloping to the inside at an angle of about 45 degrees.

Cascade, Step or Perforated Tray Aerators - This type of aerator requires less head than spray nozzles, namely about 3 pounds or less. Cascade aerators should have a slope of 1 foot on 2 or 3 and a length of flow of 3 to 10 feet, whereby the water is exposed to the air at the rate of 8 to 16 gallons per sq. ft. of area. Step or perforated tray aerators should provide a total tray area of 1 sq. ft. per 4 to 8 gpm. flow. From three to five trays or steps should be used, with vertical spacing of 8 to 15 inches. Perforation of trays should be 3/16 to 1/2 inch in diameter, spaced 1 to 3 inches center to center. Improved results are secured when coke or crushed stone having a size of 1-1/2 to 2-1/2 inches is placed on the trays, especially at iron and manganese removal plants to be discussed later.

Diffused Air Aerators - Forcing compressed air through water is more costly in power than pumping water through an aerator. If, however, the head on a gravity supply is limited and aeration is needed, then consideration should be given to the use of low pressure air compressors producing 5 to 10 pounds per sq. in. pressure with a volume of 0.01 to 0.2 cu. ft. per gallon, a basin providing a detention period of 5 to 15 minutes and a depth of 9 to 15 feet. Flocculation basins may be used for this purpose because such aeration aids in agitation. The air should be diffused through porous tubes or plates to form small bubbles having a large total surface area.

CHAPTER IV. SEDIMENTATIONS BASINS WITHOUT CHEMICAL TREATMENT

Objectives - Preliminary sedimentation, without the use of coagulants, is practiced when turbidities are excessive, so as to reduce the dose of coagulant used subsequently in the conventional manner, or also prior to slow sand filters when plain sedimentation will reduce the turbidity of the raw water to 30 p.p.m. or less, thus permitting such filters to be operated without too frequent scraping.

Design - Such basins should be designed in the same manner as those used to settle flocculated waters, except that detention periods of 12 hours or more should be provided in basins used prior to slow sand filters. Lagoons, ponds or reservoirs may serve as pre-sedimentation facilities more economically than concrete basins.

The size of slow sand filters may be kept at a minimum by providing for 24-hour operation through special design of outlet weirs of sedimentation basins. For instance, if water is to be pumped into a preliminary sedimentation basin, and if a daily pumping period of 8 to 12 hours is desirable to lower the labor costs, then the basin should have a submerged outlet weir having an elevation about a foot above the sand surface of the filters. In this way the settled water from the basin will continue to flow to the filter, after the pump is stopped, until the settling water level drops to the level of the weir. Thus, a basin with an 18-hour detention period, and with the weir located at half depth of the basin will supply water to the filter for 9 hours after the pump is stopped. Other relative elevations and capacities may be selected to meet local needs as to labor cost, power, etc. An uniform rate of filtration over the 24-hour period must be provided through the use of rate controllers or their equivalent.

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CHAPTER V. MIXING AND FLOCCULATION BASINS

Objectives - A distinction should be made between flash mixing with violent agitation for a short period of time to distribute the coagulant, and slow agitation or floccualtion for 15 to 60 minutes to favor the coagulation and flocculation or coalescence of the floc to a size favorable for subsequent sedimentation.

Effective flocculation of coagulant-treated water should be provided to insure economical and effective operation of filters. The facilities should be designed to provide the degree of agitation for a period of time determined by pilot plant or laboratory studies with the specific water to be treated with the selected coagulant.

General - Alum must be imported into many countries, but even then its use with or without lime should be considered as standard practice. The proposed use of another coagulant should be supported by appropriate data in the engineering report.

Both horizontally and vertically baffled basins are economical and readily maintained. Basins equipped with motor-driven paddles are more costly to install and operate but they provide a ready means of controlling the degree of agitation irrespective of the rate of flow of water. The size of the plant, the head available, the ability of the operators and the available budget, however, should be appraised before selecting mechanical flocculation equipment.

Chemical Storage - The space needed for the storage of chemicals should be based upon the convenience of purchase and shipment, but should provide for at least one month's supply. Alum and lime may be purchased in bags of 100 pounds net weight, which may be piled near the chemical feeders, provided the space is dry. Convenience in operating larger plants, however, is secured by the use of storage bins located above the hoppers of the chemical feeders, and connected therewith by flexible cloth tubes to permit the hoppers being filled by gravity without the production of dust. The floor above the storage bins constitutes their cover and also provides extra storage space for chemicals in bags. A hoist or block and tackle should be provided for elevating chemicals from the unloading platform to the storage space.

The properties of alum and alkalis used in coagulation of water are listed in Table I.

Chemical Feeders - A minimum of two chemical feeders should be provided, or a total of three feeders when alum and lime are used. They should be located and enclosed so as to localize dust. The use of dust control equipment is warranted at larger plants.

Chemical feeders should have a capacity above the maximum requirements, but not so large as to be inaccurate during periods of low dosage. Solution feeds are advocated for small plants. Dry feeders are less laborious to operate and are subject to more accurate, direct control and are better suited to larger plants. The type of feeder should be selected with due regard to the character of service by the manufacturer, the availability of spare parts, etc.

Solution Tanks - Tanks for alum solution should be of bitumastic enamel coated concrete, or wood. Concrete or steel tanks should be used for lime suspensions. Solution tanks should be fitted with hand operated or motor-driven revolving paddles for mixing the alum solution. Continuous mixing is necessary for maintaining a uniform suspension of lime. Float gauge, scale or sight glass should be provided to indicate the level of the solution. Solution tanks should have a sump with drain for collecting the insoluable impurities. Open drains are preferable to closed pipe. Make-up water should be discharged through a pipe terminating above the maximum level of the solution to avoid back-siphonage. The capacity of solution tanks should be greater than the maximum amount of solution used in any one shift.

Chemical Feeder Controls - The use of constant capacity raw water pumps, or individual filter-rate controllers with meters permits manual changes in the observed rate of flow of water being treated, and under these conditions manually controlled chemical feeders may be used. In other instances the chemical feeders should be of the automatic type, capable of feeding chemical proportional to the rate of flow of water being treated. Automatice shut-off controls should be provided when a filter plant and pumping station are designed to shut down automatically when tanks or reservoir for filtered water become filled. Such controls, however, should not be used to start a plant automatically because the raw water may have changed in quality during the shut-down period, so the operator should be present to adjust chemical doses each time the plant is started manually.

Automatic controls and equipment should not be used at small plants where they may not be fully understood and maintained by the operator, nor used at large plants unless they are of proven reliability and are capable of being maintained and repaired. Spare parts should be maintained to facilitate repairs and to prevent interruption in treatment.

Chemical Feed Lines - Rubber or plastic hose or lead tubing, supported at short intervals or placed in a 4 inch diameter tile pipe, are well adapted for use with alum solutions, as they are resistant to the action of acids and are easily removed for cleaning. Lime suspensions may be conducted either in rubber hoses or in wrought iron pipe which, however, should be fitted with sufficient unions and crosses with plugs at 90° bends to facilitate cleaning. Rubber hose is preferable because adhering solids may be broken loose by flexing the hose. Open channels are best for handling lime suspensions.

Flash Mixing - The rapid mixing of the coagulant throughout the water being treated should be assured by flash mixing for a short period. This may be secured by use of a baffled channel or portion of the inlet flume with velocities of 2.5 to 5.0 feet per second, a "hydraulic jump" flume with a velocity of about 10 feet per second discharging into water moving about 2.5 feet per second, or by using a small basin with a detention period of about 30 seconds where rapid mixing is secured by high-speed, motor-driven propellers or paddles. Usually detention periods are from 30 to 60 seconds.

Flocculation - Flocculation should be secured by agitation resulting from velocities between the minimum of 0.5 feet per second, which prevents the sedimentation of the forming floc, and a maximum velocity of 2.0 feet per second, above which small flocs difficult to settle are formed. Flocculation periods should be 15 to 30 minutes, the shorter periods being associated with higher velocities. There is a trend towards flocculating periods of 30 to 60 minutes, but these longer periods generally are not needed with warmer water temperatures prevailing in the tropics. Flocculation velocity and period should be established for each design, through laboratory studies of the specific water to be treated.

Baffled flocculation basins are widely used and are inexpensive. Their chief defect is the high loss of head when suitable periods are used. Furthermore the degree of agitation changes with the rate of flow of water. Around-the-end baffles are preferable to over-and-under baffles, because the loss of head need not be predicted so closely in the design, and also because the baffles may be altered readily after the plant is constructed. The use of wooden baffles is advocated to facilitate such alterations.

Helical flow basins are preferable to baffled basins, especially when the available head is limited and when the use of mechanical equipment must be kept to a minimum. These basins should be circular or square in plan and the water should enter tangentially near the bottom so as to cause helical flow to the effluent pipe located at the flowline of the tank. The entering water should have a velocity of about 1.5 feet per second to secure the required jet action causing helical flow. Successive reductions in the degree of agitation may be secured by using a series of smaller basins, reducing the jet velocity of the water entering each basin by increasing the area of each inlet opening. This area may be made adjustable by using sluice gates, or replaceable boards to close a portion of the openings. Jet velocities of 1.5, 1.2, 1.0, 0.8, 0.6 feet per second for a series of 5 basins are suggested.

The most effective, though costly, flocculation basins are those fitted with motor-driven paddles, which either revolve or oscillate. Variable-speed motors, or adjustable speed-reduction gears are advocated to permit adjusting the degree of agitation secured by peripheral speeds of paddles between 0.5 and 1.5 feet per second. The total area of the paddles should be 10 to 25% of the cross-sectional area of the basin. Either vertical or horizontal shafts are acceptable. Basins with a square plan are favorable for vertical units, but rectangular basins are more suitable for horizontal units. Equipment may be secured from a number of manufacturers. The design should be based upon the characteristics of the selected equipment, and patented details.

Care should be exercised to prevent short-circuiting of water through such basins, such as by having inlet openings near the surface and the outlet openings near the bottom. An economical arrangement is the use of one large basin, divided by a perforated stilling wall into two units, one serving as the flocculating basin with mixing paddles, and the other as the sedimentation basin. This arrangement also has the advantage of flexibility because the perforated wall may be relocated, if made of wood, so as to readjust the relative length of the flocculation and sedimentation periods.

The size of the effluent pipe or conduit leading to the sedimentation basin should be sufficient to prevent velocities of flow over 1.5 feet per second, so that the formed floc will not be destroyed.

CHAPTER VI. SEDIMENTATION BASINS WITH CHEMICAL TREATMENT

The design of such units involves many factors, such as number of basins, length, width, depth, velocity of flow, detention time, volume of sludge storage, method of sludge removal, inlet and outlet arrangement and the coagulating characteristics of the specific water to be treated. Undue importance should not be attached to theoretical detention period, because the inlet and outlet arrangement, the length to width ratio and depth of basin determine the effective, average flowing-through period, which usually is considerably shorter than the theoretical detention period. Best results are secured with long, narrow and relatively shallow basins, with the inlet at one end and the outlet at the other one, so that the direction of flow is not reversed.

At least two basins should be provided to insure uninterrupted operation while one is being cleaned.

The detention period should be at least four hours, based upon the maximum capacity of the plant, so as to provide some factor of safety for periods of poor flocculation.

Basins should be proportioned so that their length is at least twice their width and preferably more. Depths of 10 to 15 feet provide a distance-of-travel of the settling floc reasonably short, so as to insure the floc reaching the bottom before the effluent end of the basin is reached by the flowing water. Sludge storage space should be provided, as discussed later.

Inlets should be so designed and proportioned that the flocculated water will not be unduly agitated by any weir action or turbulence, otherwise the floc formed previously will be broken up. The influent pipe or flume should be proportioned to provide a velocity not greater than 1.5 feet per second, and it should discharge behind a submerged weir or perforated baffle or into a perforated flume to distribute the water as uniformly as possible across the inlet end of the basin. In addition a stilling or diffuser wall, perforated or slotted so as to secure velocities through the slots of 0.4 to 0.8 feet per second should be provided to minimize eddies and encourage uniform flow through the basin. These velocities necessitate the total area of the slots being restricted. Improved dissipation of velocity head is secured by using expanding slot openings with surfaces having an angle of 15° from the direction of flow. The selection of the lower value of 0.4 feet per second, when both basins are in use, will prevent velocities higher than the upper value when only one basin is in use. Frequently the slots have been equally spaced throughout the full depth of the wall, or within the upper 75% of the wall's depth. More recently, unperforated walls have extended as baffles for two-thirds the depth, so as to divert the incoming water to the lowest one-third of the tank depth. This, however, does not prevent eddy-currents, nor secure uniformity of flow throughout the full width of the tank. The advantages of both procedures may be obtained by placing the slots in the lower two-thirds portion of the baffle, whereby surface short-circuiting is avoided, and resistance to flow through the restricting slots spreads the flow across the basin width. The stilling wall should be 3 to 10 feet from the inlet end, depending upon the size of the basin. Consideration also should be given to fitting large basins with several "training walls" extending at right angles from the stilling wall for about 10 feet to minimize cross-eddies.

Overflow weirs should extend at least across the effluent end of a basin, and for such distance along each side of the basin as may be considered desirable to provide an adequate total length of weir. Submerged weirs were favored in the past as they eliminated turbulence. The trend to great weir length, however, required true weir action to insure uniform flow throughout collecting troughs or conduits. The maximum weir loading advocated in the past was 40,000 gpd per ft. of length, but values of 8,000 to 16,000 gpd per ft. are preferable for normal plant operation. More certain weir action may be secured with a series of V-notched weirs about 2-1/2 inches deep and on centers 6 to 12 inches apart, than with a long length of level weir, having a very low depth of flow.

Sludge storage space should be provided in accordance with the characteristics of the raw water and the anticipated average dose of coagulant. An additional 1/2 to 1 foot to the depth of a basin should provide adequate storage with colored waters of moderate turbidity.

Sloping the bottom 1 on 12 towards a center gutter, which in turn slopes to one end of the basin facilitates the removal of sludge, but small basins may have flat bottoms. Provisions should be made for flushing of sludge with hose streams.

The settled water conduit leading to the filters should be proportioned so as to avoid a velocity in excess of 1.5 feet per second, and bends should be designed to avoid turbulence which would tend to destroy the floc remaining in the settled water.

Settling basins may be covered to exclude light and thus prevent the growth of algae on the walls.

<u>Summary</u> - Properly formed floc should settle at rates in excess of 2.0 feet per hour, or 8.0 feet in 4 hours. Selecting a minimum, anticipated rate of 10.0 feet per 4 hours, the minimum depth of a basin should not be less than 10.0 feet, otherwise the water will reach the outlet weir before all floc has settled. The vertical component of the rate of flow of water cannot exceed this value of 10 feet in 4 hours, which is equivalent to 360 gal/day/sq. ft. of surface area of a basin. The rate of 320 gal/day/sq. ft. is a conservative norm. The design of a basin is based upon: a) the quantity of water to be treated; b) the selected detention period; and c) the selected surface overflow rate. The attached nomogram (Figure 1) is convenient for determining the required dimensions of sedimentation basins.

For example, suppose a filter plant of 1.0 mgd capacity is under design, and that a detention period of 4.0 hours and a surface overflow rate of 320 gal/day/sq. ft. have been selected as the basis of design for the sedimentation basins. Then the nomograms indicate from the rate

Nomographic Chart for the Design of Settling Tank Capacities and Dimensions

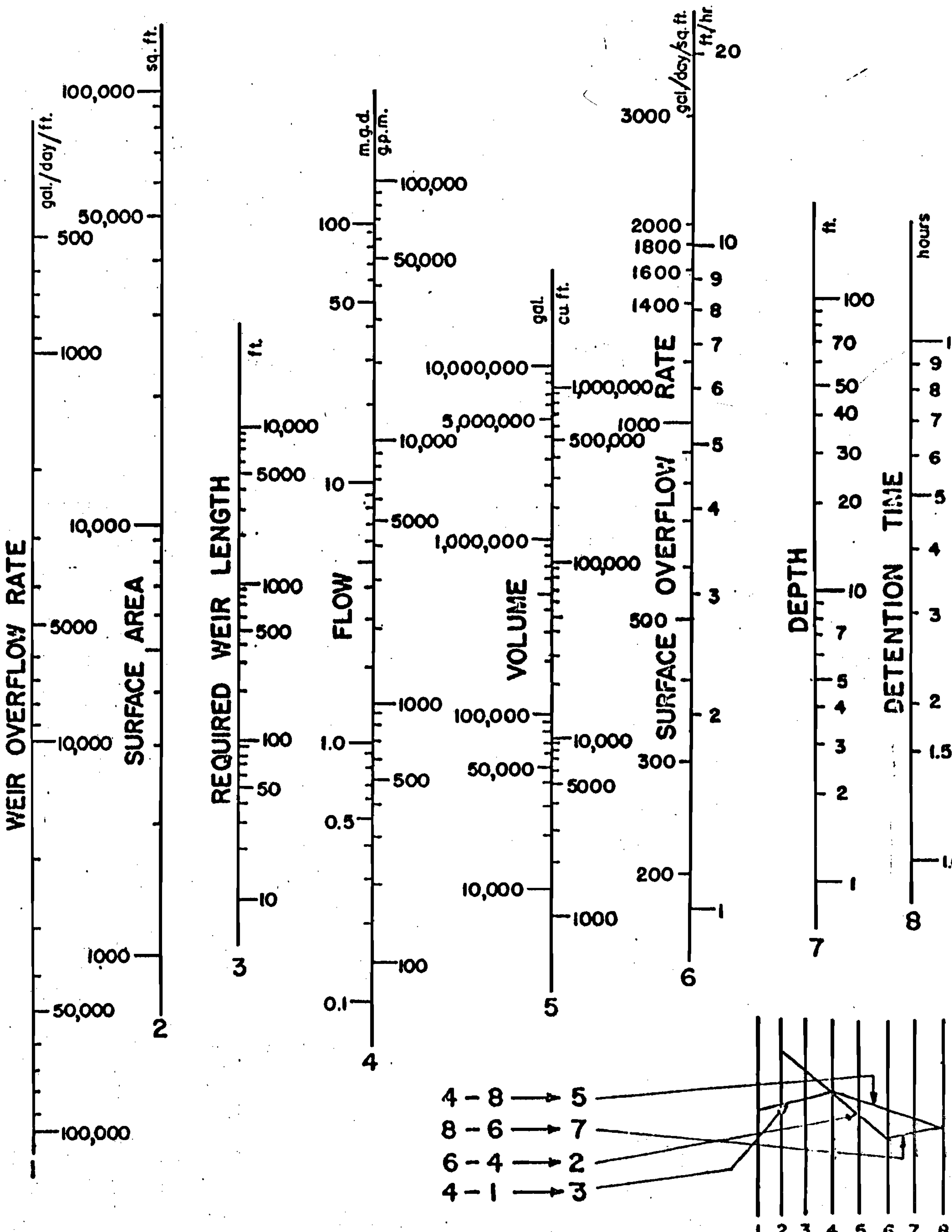

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Figure 1

of flow and detention time that the volume is about 132,000 gallons. The surface overflow rate and the rate of flow gives a surface area of 2,500 sq. ft. The detention time and surface overflow rate give a depth of about 9 feet for the portion of the basins above the sludge storage space. Ordinarily added depth of sludge storage need be only 1 to 3 feet for colored water of moderate turbidity, so the basins used in this example could be given a total depth of 11 feet.

Bearing in mind that at least two basins should be used and that each unit should have a length of two to three times its width, the total surface area of 2,500 sq. ft., requires two basins of 1,250 sq. ft. area. A length of 63 ft. and a width of 20 ft. provides 1,260 sq. ft. and a length-width ratio of 3 to 1.

The width of the two basins of 40 feet gives a weir overflow rate from the nomogram of 20,000 gpd/ft., which is acceptable. A slower velocity of approach of the water, however, may be secured by using a double-edged weir trough, located 3 to 5 feet from the outlet end of the basin, thus providing twice the weir length, or 80 feet, and a weir overflow rate of 10,000 gpd/ft.

Solid Contact Basins - Upward-flow, solid-contact basins were developed for the lime-soda softening process, and later were used for the clarification of turbid waters. In general the coagulation of water in the presence of previously formed floc or "sludge blanket" is sound in principle. Such basins have been successful in the lime-soda softening process, even when the total period for flocculation and clarification has been only one hour, because of the high specific gravity of the calcium and magnesium carbonate floc. Alum floc, however, is much lighter, especially with colored waters of low turbidity. Therefore, periods of 1-1/2 to 2 hours are needed for this type of clarification basins, even when they are applicable.

Furthermore, water of low turbidity, or colored waters, require the use of proportionally greater alum doses to secure a floc which will settle readily in the upward-flowing water in the clarification zone of the basins, unless floc formation is aided by the more complex coagulant aid or activated silica treatment. Therefore, the saving due to small size of such basins is counterbalanced by the higher cost of mechanical equipment and patented appurtenances and by higher chemical costs, as compared with conventional basins. In addition, more skilled operation is required to insure proper operation and complete and rapid coagulation at all times. Solid-contact basins, therefore, are recommended only for lime-soda softening, or under special conditions, where raw water characteristics, size of plant, adequacy of technical control and other local factors justify a special engineering design including such basins.

The manufacturers of solid-contact basins have developed patented details including proportions of the units, so standardizing of design is not feasible. Designing engineers, therefore, should determine the total detention period required for the effective treatment of a specific water and then select the size of unit having a fluid capacity sufficient to provide this period and having patented design characteristics considered best suited to local conditions. The engineering report should present data supporting any plans including such basins.

TABLE I

"PROPERTIES OF COAGULANTS AND ALKALIS USED IN COAGULATION"

CHARACTERISTICS AND RECOMMENDATIONS	ALUMINUM SULFATE	CALCIUM HYDROXIDE	CALCIUM OXIDE	SODIUM CARBONATE
FORMULA	$Al_2(SO_4)_3 \cdot 14H_2O$ (Approx.)	$Ca(OH)_2$	CaO	Na_2CO_3
COMMON NAME	(Alum, Filter Alum, Pickle Alum, Sulfate of Alumina)	(Hydrated Lime, Slaked Lime)	(Quick Lime, Burnt Lime, Chemical Lime, Unslaked Lime)	(Soda Ash - 58%)
USE	Coagulation, at pH 5.5 to 8.0 Sludge conditioner.	Coagulation, softening, pH Adjustment	Coagulation, softening pH Adjustment	Water softening pH Adjustment
AVAILABLE FORMS	Ground, Rice, Powder, Lump	Powder	Pebble, Crushed, Lump, Ground	Dense crystals, Light Powder Extra Light
CONTAINERS AND REQUIREMENTS	Bags-100, 200 lb, Bbl-325, 400 lb Drums-25, 100, 250 lb, Bulk-C/L	Bags-50 lb, Bbl-100 lb, Bulk-C/L (Store in dry place)	Moisture proof bags-100 lb, Wood bbl Bulk-C/L (Store dry-Max. 60 days; keep container closed)	Bags-100 lb, Bbl-100 lb, Drums-25, 100 lb, Bulk-C/L
APPEARANCES AND PROPERTIES	White crystals. Low, even solubility 1% Soln.-pH 3.5	White, 200-400 mesh powder, free from lumps. Caustic, irritant dusty Sat. Soln.-pH 12.4	White (light gray, tan) lumps to powder. Unstable, caustic, irritant Slakes to hydroxide slurry evolving heat. Sat. Soln.-pH 12.4	White, alkaline 1% soln.-pH 11.2
WEIGHT lb/cu ft (Bulk Density)	60 to 75 (Powder is Lighter)	35 to 50 To calculate hopper capacity, use 40	55 to 70 To Calculate hopper capacity, use 60	Dense 65 Light 40 Extra Light 30
COMMERCIAL STRENGTH	17% Al_2O_3 (Min.).	$Ca(OH)_2$ 82 to 99%	70 to 96% CaO (Below 88% is poor quality)	99.2% Na_2CO_3 58% Na_2O
SOLUBILITY IN WATER gm / 100 ml	60.0 ( ) 0°C 65.3 ( ) 10°C 71.0 ( ) 20°C 78.8 ( ) 20°C	0.18 ( ) 0°C 0.16 ( ) 20°C 0.15 ( ) 30°C	Reacts to form $Ca(OH)_2$ (See solubility) of $Ca(OH)_2$ (See Footnote 4)	7.0 ( ) 0°C 12.5 ( ) 10°C 21.5 ( ) 20°C 38.9 ( ) 30°C
BEST FEEDING FORM	Ground or rice about 60 lb/cu ft Powder is very dusty, arches in hoppers and is troublesome	Finer particles sizes more efficient, but more difficult to handle and feed (See Footnote 3)	3/4"- pebble Lime, Crushed lime to pass 1-in ring (fine included) Ground lime arches and floods in feeders; also burst bags in storage	Dense
CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	0.6 lb/gal-Max. Dissolver detention time 5 min. - Min.	Dry Feed 0.6/lb/gal-Max Slurry 1.16 lb/gal, i.e. 10% solution - Max.	2.5lb per gal (Range from 1.75to 3.0 lb/gal) dilute after slaking to 1.16 lb/gal (10% soln.) Max.	Dry feed 0.3 lb/gal for 10 min. detention time 0.6 lb/gal for 20 min. Solution Feed 1.25lb/gal.
TYPES OF FEEDERS	GRAVIMETRIC Loss-in-Weight 1 to 1000 lb/hr Belt - 20 lb/hr, up Disc - up to 20 lb/hr Universal - 10 lb/hr, up	GRAVIMETRIC L-I-W- 1 to 500 lb/hr Belt - 50 to 2000 lb/hr VOLUMETRIC Disc - up to 20 lb/hr Universal - with small hopper Rotolock - with large hopper LIQUID Precision, Sampler, Pump, Rotodip Prop. Weir tank	GRAVIMETRIC Loss-in-weight - Up to 2000 lb/hr Belt - Up to 10,000 lb/hr VOLUMETRIC Universal - Up to 6000 lb/hr	GRAVIMETRIC Loss-In-Weight - Up to 500 lb/hr Belt - 20 lb/hr, up VOLUMETRIC Disc to 20 lb/hr, up Universal - 10 lb/hr, up SOLUTION Precision - To 32 gph Prop. Pump - To 57 gph
ACCESSORY EQUIPMENT REQUIRED	Dissolver Scales for volumetric feeders Dust Collector	Hopper agitators Non-flood rotor under large hoppers Dust Collectors	Hopper agitator and non-flood rotor for ground lime Recording Thermometer Water proportioner Lime slaker High Temperature safety cut-out and alarm	Rotolock for light forms to prevent flooding Large dissolvers Bin agitators for light form
SUITABLE HANDLING MATERIALS	DRY - Iron, steel, concrete WET - Lead, rubber, Dur iron, asphalt cypress, stainless steel type 316	Rubber hose, iron, steel, asphalt concrete; No lead	DRY - Iron, steel WET - Rubber, ceramics	Iron, steel, rubber hose

CHAPTER VII. FILTERS

Objectives - Sand filters are not merely "strainers" for removing suspended solids larger than the spaces or pores between the sand grains. Inasmuch as colloidal clay and coloring matter and bacteria are smaller than these spaces or pores, their removal involves complex processes. Filter design, therefore, involves a consideration of many factors favoring these processes.

Slow sand filters should be designed so that water flows at a slow rate through fine sand so that coarser, suspended solids are caught on or near the surface of the bed to form a very fine porous layer having a large total surface area of channels or pores, whereby absorption of impurities in the layer and in the underlying sand is facilitated. This requires a large area and slow rates to insure surface contact and absorption.

Rapid sand filters should be designed to receive coagulated and settled water, whereby colloidal material and bacteria are absorbed on the gelatinous floc and are removed with the floc. Adequate pre-treatment, therefore, is essential to effective rapid sand filtration.

Serious engineering study should be given to the relative advantages and disadvantages of slow sand filters versus rapid sand filters for each specific plant under design. Once this choice has been made, the design should be based upon the inherent characteristics of the type of filter involved.

SLOW SAND FILTERS

The advantages of this type of filter may be summarized as follows:

1) No need for coagulation facilities;
2) Equipment is simple;
3) Suitable sand is more readily secured;
4) Supervision is simple; and,
5) The effluent is less corrosive than chemically treated waters.

The disadvantages may be summarized as follows:

1) Large area required, with correspondingly large structure and volume of sand and higher structural costs;
2) Less flexibility in operation;
3) Not economical with raw waters having turbidities over about 30.0 p.p.m. for prolonged periods, unless preliminary plain sedimentation will produce such turbidities in the settled water;
4) Lower effectiveness in removing color; and,
5) Poor results with water of high algae content, unless pre-treatment is practiced.

In general the advantages of these filters justify their use for small plants not under technical supervision, where relatively clear surface waters are to be treated.

Pre-treatment - Ordinarily, slow sand filters are used without pre-treatment. Plain sedimentation prior to filtration may reduce the turbidities of many raw waters below 30.0 p.p.m. and thus permit their economical filtration without the cost and technical complexity of coagulation. See Chapter IV - "Sedimentation Basins Without Chemical Treatment".

Pre-chlorination of water to be filtered through slow sand filters formerly was considered undesirable because the "slime" organisms on the sand grains, held to be essential for effective filtration, would be destroyed. Experience has demonstrated, however, that effective filtration, with longer filter runs, can be secured with pre-chlorination, so as to destroy these organisms, as well as to control algae growths on the surface of the sand. This is of great practical significance when filter runs are influenced more by the clogging effect of such organisms than by suspended solids.

Consideration also should be given to pre-treatment with copper sulphate, when algae are sufficiently prevalent to seriously reduce the filter's run. Inasmuch as a continuous dose of this chemical can be less than that commonly used periodically, the chemical may be applied by a small solution feeder at a dose of 1-1/4 pound per million gallons. Such pre-treatment is aided when a pre-sedimentation basin is available, but application directly to the filter influent is acceptable.

Design Details - At least two filters should be provided, and three beds are desirable when the applied water is such as to produce short filter runs and require relatively frequent scraping.

The filters should be covered to minimize algae growths. Head-room of 6 feet should be provided to facilitate scraping. The cover should be provided with sufficient number of openings to provide daylight for scraping and to facilitate removal and replacement of the sand.

Rates of filtration from 2.0 to 5.0 million gallons per acre per day are acceptable. The rate for a specific plant should be selected with due regard to the character of the raw water and the desired period between scraping the filters. Ordinarily a rate of 4.0 m.g.a.d. results in a satisfactory compromise between economy in structural costs and in operation.

Each filter should be fitted with a loss-of-head gauge. Automatic rate controllers provide convenient control, but the loss-of-head builds up so slowly with such filters that manual control is feasible, provided an orifice, a venturi meter or some other metering device is provided. Consideration should be given at small plants to the use of an adjustable float valve in a weir or orifice box to serve as a rate-controller. A float-operated, butterfly valve may be used, as described later.

Slow sand filters should not be operated under a negative head, otherwise "air-binding" will occur. Therefore, the depth of water over the sand limits the available loss-of-head and in turn the length of filter runs. Accordingly the depth of water should not be less than 4 feet and consideration should be given to six feet of head-room under any filter cover to provide five-foot depth of water and one-foot of free-board.

Each filter should be equipped with a main drain and sufficient number of lateral underdrains to collect the filtered water.

Tile pipe with open joints commonly are used. They should be so spaced that the maximum velocity of flow does not exceed 0.75 feet per second. The maximum spacing of underdrains should not exceed 10 feet and they should not extend any closer than 2 feet from the side walls so as to prevent water flowing in the space between the sand and concrete wall and thence into the gravel without passing through sand.

Filter gravel should be so graded as to prevent the penetration of sand and yet provide for the free flow of water towards the underdrains. The following minimum depth of graded gravel will be acceptable:

Six-inch layer of gravel passing a three inch screen but held on 3/8 inch screen;
Two-inch layer passing a 1 inch screen but held on a 3/8 inch screen;
Two-inch layer passing a 3/8 inch screen but held on a 3/16 inch screen.

The gravel should be placed over the underdrains, but should not be placed closer than 2 feet from the side walls of the filter, so that only sand will rest on the filter bottom in the two-foot zone along the sides of the filter, where there are no underdrains, for the reason noted above.

Filter sand may be available from local sources, but usually it must be washed before use to remove clay, foreign matter and very find sand grains. Washing may be accomplished by the use of homemade equipment, or by special equipment using the jet action of a stream of water.

The size of sand for slow sand filters preferably should be selected with due regard to the character of the raw water and the intended rate of filtration, lower rates and fine sand being used when bacterial pollution is more serious, or when the suspended solids are finely divided. The "effective size" should not be less than 0.2 mm nor greater than 0.4 mm, and usually should be about 0.3 mm. The sand need not be as uniform as with rapid sand filters, but the "uniformity coefficient" should be no greater than 2.5.

PRESSURE FILTERS

Pressure sand filters are not ~~acceptable~~ very satisfactory for the filtration of public water supplies previously coagulated for the removal of color, turbidity and bacteria, although they are satisfactory for zeolite softening or iron and manganese removal not associated with bacterial removal. The reasons supporting this policy may be summarized as follows:

1) Difficulty of coagulation and sedimentation under pressure, unless double pumping is practiced to permit the use of open flocculating and sedimentation facilities;

2) Difficulty of applying chemicals under pressure;

3) Sand bed is not subject to inspection while in operation or while being washed;

4) Difficulty of properly collecting wash water due to shape of filter tanks; and,

5) Difficulty of inspecting, cleaning or replacing sand, gravel and underdrains.

RAPID, GRAVITY SAND FILTERS

The various units of rapid sand filters should be designed so as to be coordinated into the whole plant in accordance with the basic plan. It is especially necessary for the character of the water, the proposed rate of filtration, the size of the plant and the anticipated reliability of the operators to be considered in relationship to each other. In other words, conservative design should be followed when factors of safety must be provided, to compensate for anticipated difficulties of treatment or less skilled operation. Reliability of operation and of power and other local factors should be considered in selecting the daily period of operation and the capacity of storage for filtered water. Consideration should be given in the design of small plants to daily periods of operation short enough for single-shift control, even though this requires a larger structure.

Rates of Filtration - The normal rate of filtration should be 2 g.p.m. per sq. ft. of filter area. Any proposed rate in excess of this should be considered in the light of all aspects of the plant, and should be supported by appropriate data in the engineering report.

In general, filters should be designed so as to provide means for removing residual floc from properly pre-treated water, and also to serve at times of poor or delayed coagulation as "contact beds" to insure the completion of coagulation and clarification of the water before it reaches the bottom of the filter. Rapid sand filters, however, should not be expected to function continuously to clarify improperly pre-treated waters. Lower rates of filtration, finer sand and greater depth of sand favor coagulation by contact in filters, and hence provide a factor of safety, but require a larger filter area and more frequent washing of the filters.

The following details should be considered in connection with filter rates, in the light of local conditions and in their interrelationship:

a) The present minimum, average and maximum consumption of water in the municipality served by an existing water supply system;

b) The designed capacity of the plant;

c) The foreseeable future consumption of water and the feasibility of meeting this demand later by enlarging the plant, but without exceeding the higher rate of filtration under consideration;

d) The quality of the raw water and the degree and rapidity of the fluctuation in quality;

e) The ease of coagulating, settling and filtering the supply;

f) The training and reliability of the operator and the resulting reliability of the pre-treatment in assuring an effectively coagulated and settled water irrespective of changes in the quality, temperature, etc., of the raw water;

g) The size and depth of filter sand; and,

h) The effectiveness and reliability of disinfection.

At least two filter units should be provided, and consideration should be given to designing each unit to take the full load when only two units are provided, so that one unit may be repaired without closing down the plant. The filter structure should have an adequate depth to provide at least 4 feet of water over the sand so as to discourage air-binding and insure reasonable length of filter runs. The minimum total depth should be at least 8-1/2 feet and preferably 10 feet.

The settled water should be introduced in a manner to avoid turbulence, for example in a stream directed at the open ends of wash troughs, so the water does not impinge against a concrete wall.

General - The filter structure should be designed as an unit, with due consideration to the interrelationship between the quality of the raw water, the range in temperature of the raw water, the selected rate of filtration, the depth and size of sand, the rate of flow of wash water to secure 50% of sand expansion, and the height of the wash water troughs above the sand. These factors are discussed separately below. Their integration is illustrated by the basis of design of the small plant described in the Appendix. In summary, the logical procedure is as follows:

1) Select the rate of filtration in the light of quality of raw water, anticipated effectiveness of operation, etc., as described previously.

2) Select the depth and size of sand most effective with the design rate of filtration.

3) Determine from Figure 2 the required rate of flow of wash water, based upon the maximum water temperature and the effective size of the sand, to give sand expansion of 50%.

4) Design the wash water troughs so that their lower surface is slightly above the expanded sand during filter washing, with a width and depth sufficient to provide the needed capacity. The upper edge of the troughs should not have an elevation above the sand much greater than the distance the wash water rises in one minute. To prevent this distance being exceeded, select a wider but more shallow trough, with its lower surface just above the expanded sand level.

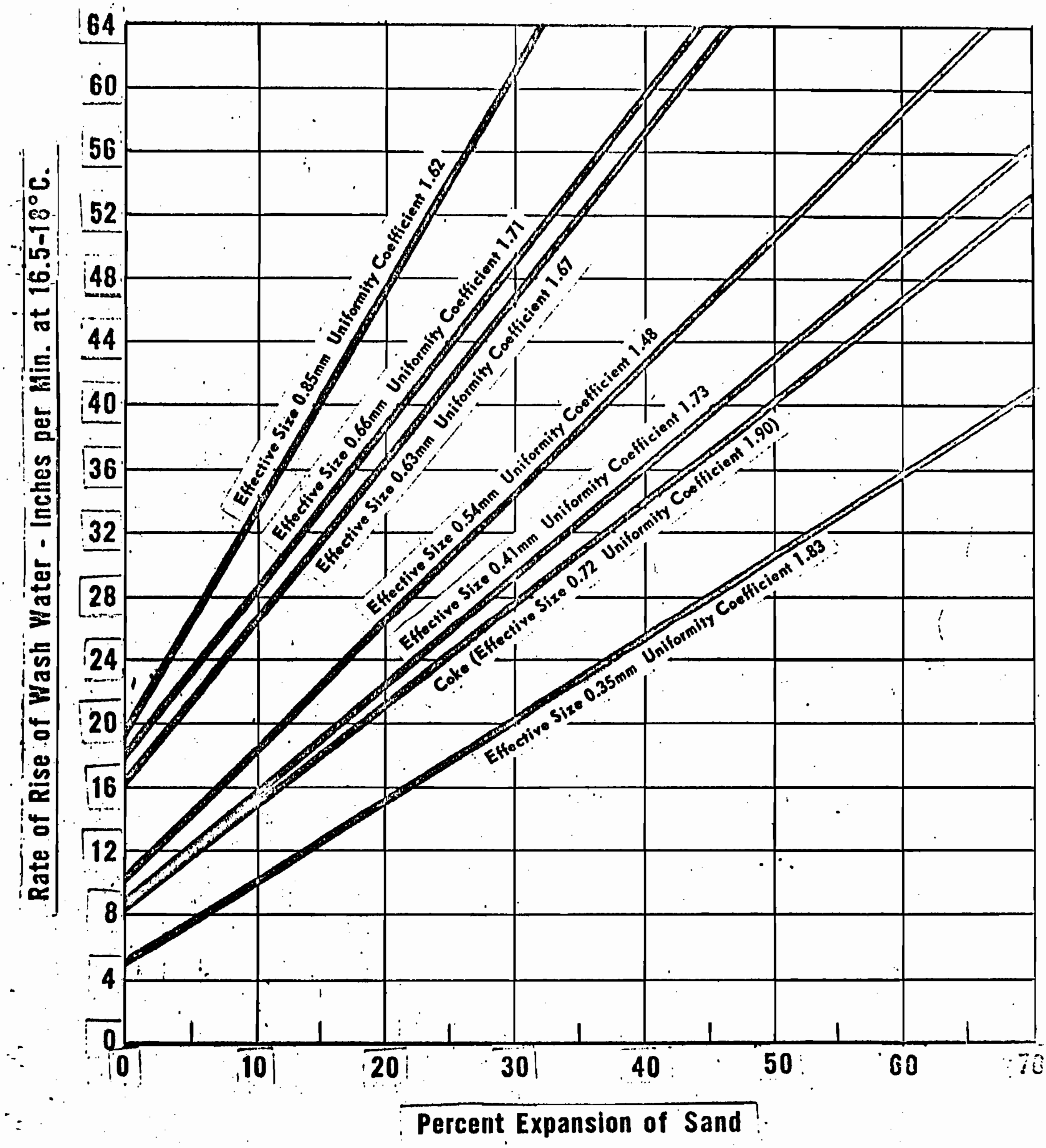

Figure 2

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Deep filters basins are recommended so as to insure long filter runs, without serious negative heads developing.

Nomograms Figure 3 and Figure 4 are convenient in coordinating filter dimensions with designed rates of flow.

Filter Sand - Filter sand should be selected with care, with due regard to the anticipated effectiveness of pre-treatment, the rate of filtration and depth of sand. When conditions warrant the selection of a unit rate of filtration of 2.0 g.p.m. per sq. ft., the sand may have an effective size in the range of 0.40 to 0.50 mm. to secure reliable filtration with reasonable filter runs. If higher rates of filtration are deemed feasible and effective pre-treatment is assured, the sand should be coarser, having an effective size between 0.50 and 0.70 mm., so as to ensure reasonable filter runs. A factor of safety is provided by a sand depth of 30 inches. The minimum depth of sand should not be less than 24 inches. The sand should be as uniform as feasible, the uniformity coefficient being no greater than 1.8.

The filter sand preferably should be supported by a 3-inch layer of torpedo sand having an effective size between 0.80 and 2.0 mm., placed over the gravel.

Gravel - Gravel preferably should be rounded material and not crushed stone. The depth and grading of gravel should be selected in accordance with the type of filter bottom and strainer system used. The depth should be between 15 and 24 inches, except with filter bottoms or special strainers serving the function of the lowest layer of coarse gravel, when the depth can be reduced. No gravel is needed with filters equipped with porous filter plates which directly support the sand.

The following range in sizes and depths of gravel are suggested for consideration:

Range in size	Range in depth
2-1/2 to 1-1/2 inches	5 to 8 inches
1-1/2 to 3/4 "	3 to 5 "
3/4 to 1/2 "	3 to 5 "
1/2 to 3/16 "	2 to 3 "
3/16 to 3/32 "	2 to 3 "
Total depth	15 to 24 inches

Filter Bottoms and Underdrains - Filter bottoms and strainer systems should be designed so that most of the loss-of-head occurs in the strainers or openings and not in the manifold and laterals, so as to ensure an even flow of wash water and rate of filtration throughout the filter area. The ratio of the total area of the openings to the filter area preferably should be about 0.003 to 1.0, except with special bottoms. For instance, a filter with an area of 100 sq. ft. would be provided with underdrain openings having a total area of 0.3 sq. ft. The total cross-sectional

Figure 3

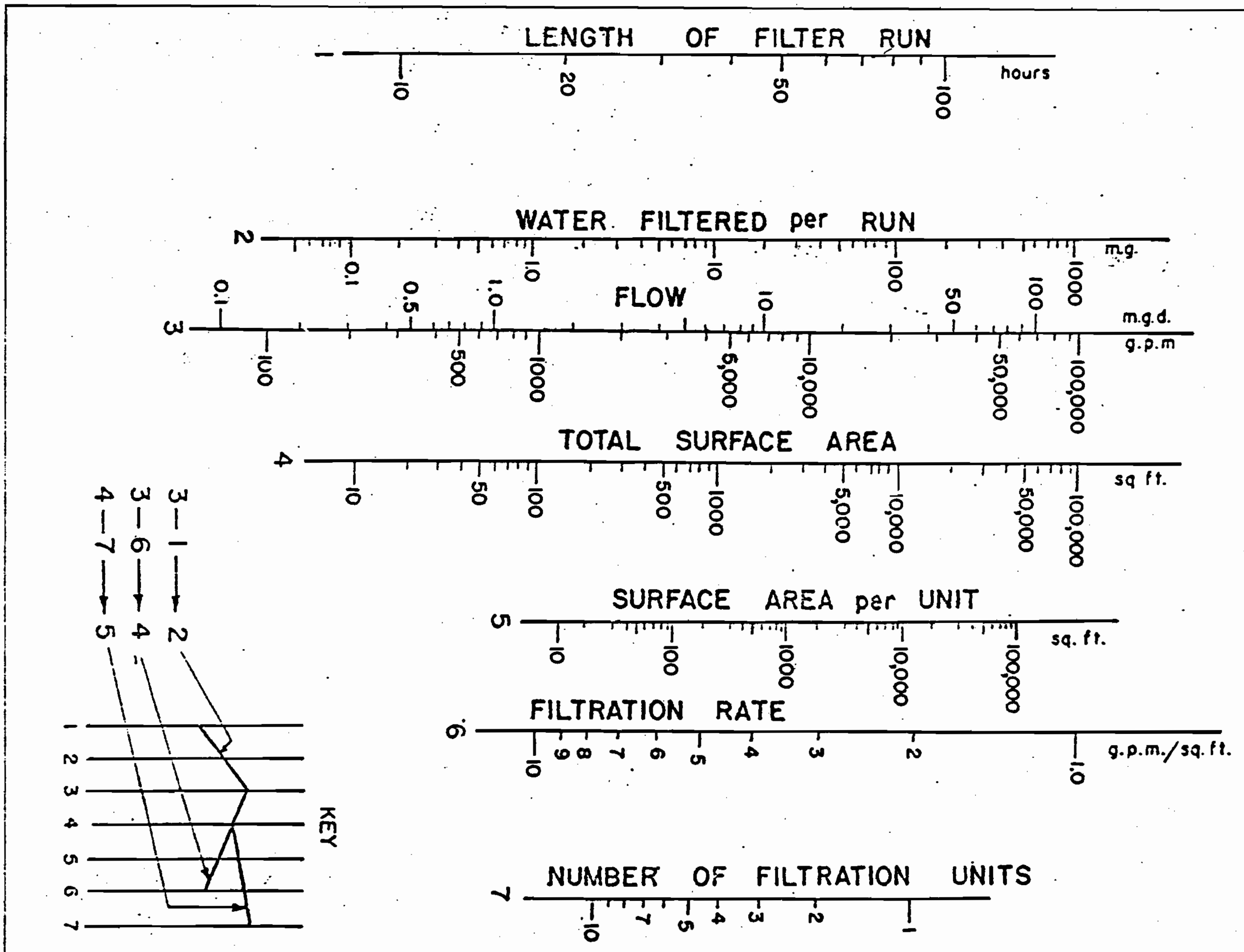

Figure 3: Rapid Sand Filters

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Figure 4: Rapid Sand Filters

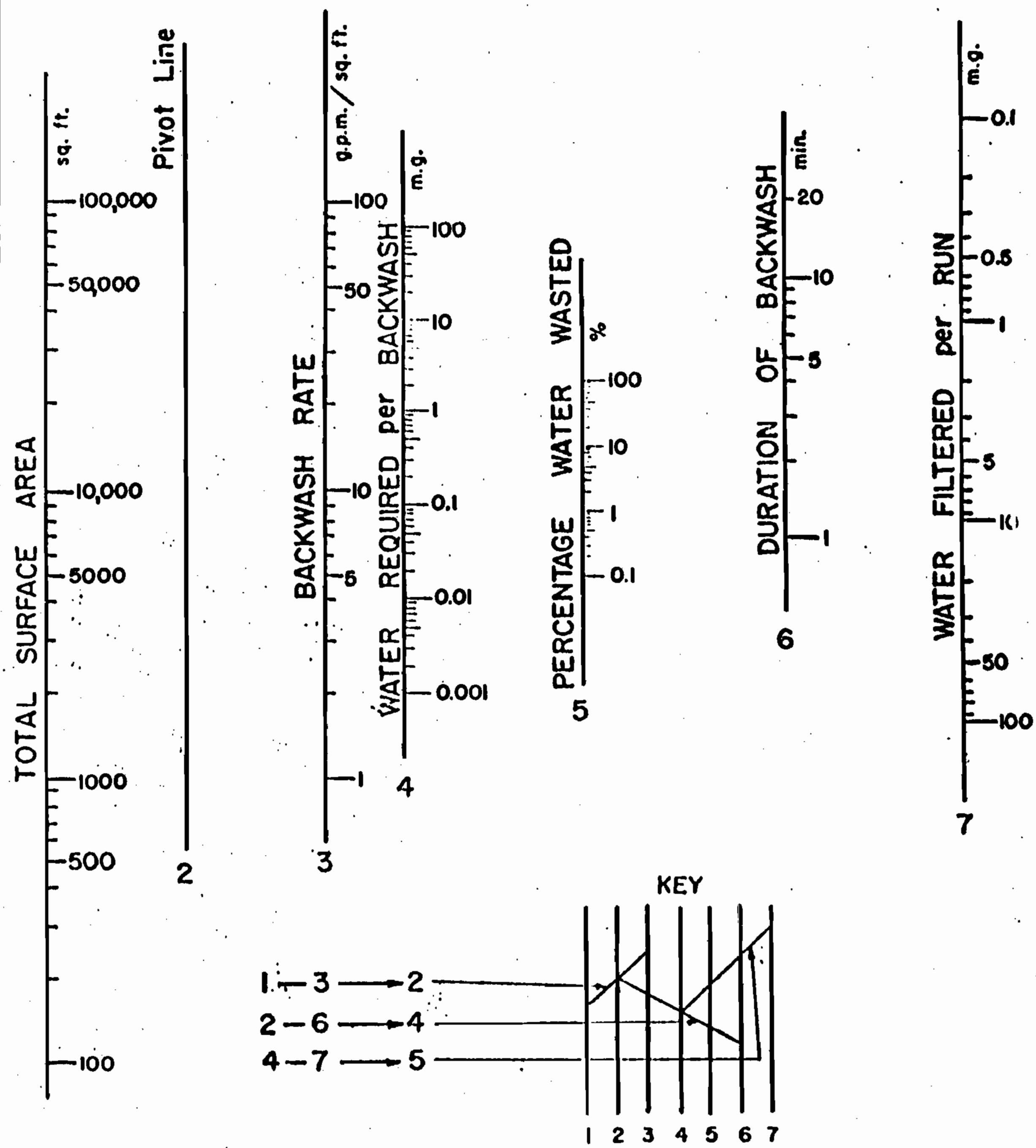

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Figure 4

area of the laterals should be about twice the total area of the strainer or the lateral openings; and the cross-sectional area of the manifold preferably should be 1-1/2 to 2 times the total of that of the laterals.

The well-established, perforated pipe underdrains also are acceptable. These may be of brass, cast iron or cement asbestos, the latter, with brass nozzles to prevent erosion of the openings, being very satisfactory.

False bottoms of concrete with porcelain or metal strainers spaced about 8 inches apart, simplify the distribution of wash water to the lower end of each strainer. An alternate is to make the false bottom of square sections of porous carborundum, fastened to stainless steel or concrete supports.

Caution should be exercised in the use of porous plate filter bottoms when coagulation may be faulty or with iron-bearing waters, or with lime-soda softening, because of the likelihood of clogging the pores of the plates.

The underdrain systems of filters equipped with "air wash" consist of pipe laterals fitted with special strainers, with their lower end extending into the lateral and with a small opening in the strainer just inside the top of the lateral. This permits compressed air to be forced into each strainer in controlled amounts, when the lower end of the strainer is submerged in water, and also permits the larger volume of wash water to enter the lower end of the strainer, when agitation with air is followed by the "water wash".

<u>Filter Washing Facilities</u> - The earliest procedures for washing rapid sand filters consisted of one of two methods. Firstly, mechanically revolving rakes were used in circular filters to agitate the sand while wash water was forced upward through the filter to separate the floc and sediment from the sand grains. Secondly, compressed air was used to secure agitation. The second procedure still is used with Candy filtering equipment of England, but high velocity water wash alone now is used in the U.S.

<u>Air - Water Wash System</u> - The patented Candy system is manufactured to provide the desired capacity, and included the integrated units of special underdrain system with nozzles, special facilities for collecting wash water, air compressor, etc. Inasmuch as thorough agitation is secured by the use of compressed air, the amount of wash water supplied is only 5 gpm per sq. ft. of filter area.

Unfortunately the old practice of using wash water rates of 12 gpm/sq/ft., or 24 inches per minute rise, continues to be used. This rate, however, was established when fine filter sand was used and in areas where cool to cold waters prevailed and before experience disclosed the basic importance of effective washing. Difficulties with mud balls, clogged areas, sand shrinkage and cracking resulted in the greater attention now being given to the maintenance of filter sand in a clean condition. This disclosed the importance of water temperature and resulted in the use of "sand expansion" as a measure of the amount of wash water needed, thus compensating for the influence of the lower viscosity of warm water. Current practice is to base the hydraulics of wash water tank and piping on 50% sand expansion, so that a factor of safety would be available. Also the trend is towards the use of coarser sand. As the result, still higher wash water rates are being designed for, with a control valve to permit the operator to select the rate best suited for conditions from day to day.

Figure 2 shows the relationship between 50% expansion, sand size and required rate of rise of wash water. The appendix shows an alternate procedure for computing the required rate of flow of wash water, as applied to a small plant in Brazil. Both of these procedures give values higher than past practice. For instance, with an expansion of 30% and a sand with an effective size of 0.41 mm, the required wash water rise is shown to be about 30 in./min. The rate for 50% expansion and sand 0.54 mm. effective size is about 47 in./min., all of these values being needed with water temperatures of about 20° C. In summary, modern practice in the U. S. is to design wash water systems to produce rates up to 36 in./min. (Allowances for warm water temperatures in the tropics justifies the use of maximum rates up to 48 in./min.)

A wash-water pump, or a wash-water tank together with a small pump capable of filling the tank in less than eight hours, should be provided, with a capacity sufficient for washing one filter for at least 10 minutes at the designed rate of flow of wash water. Larger pumps are needed when more than two filters units are used, so that they may be washed in sequence without undue delay. The bottom of the tank should be at a sufficient distance above the top of the wash water troughs to provide the head needed to produce the maximum rate of flow of wash water used in the design. (See Appendix) A wash water flow regulator, or manually controlled butterfly valve should be installed on the main wash water pipe, so the individual wash water valves on the filter units can be opened wide each time a filter is washed. Only filtered water should be used to wash filters.

Economy may necessitate the securing of wash water from the force main leaving a plant, rather than from a wash water tank or special pump taking suction from the filtered water reservoir. This practice is satisfactory when the capacity of the main and elevated storage tank or reservoir used for distribution system storage is adequate to supply the required volume of wash water without unduly reducing the pressure in the distribution system.

<u>Wash Water Troughs</u> - Wash water troughs should be spaced so each serves the same unit of filter area. The maximum <u>horizontal</u> travel of wash water preferably should not be over 3 feet, that is the edge of a trough next to a wall should be spaced 3 feet from the wall and adjacent troughs should be 6 feet apart in the clear. The bottom of the troughs should be above the elevation of the expanded sand during the wash process. Inasmuch as sand depth of 30 inches is becoming standard, the lower surface of the trough should be not less than 17 inches above the sand surface to permit 50% sand expansion without loss of sand.

The width and depth of a trough is determined by the required capacity to carry off the wash water throughout its length, with the maximum head of water inside the trough being less than its total depth, so as to insure free fall of water into the whole length of the trough. The rate of flow of wash water is determined, as noted above.

Economy may be obtained with large filter units, having long troughs, by sloping the invert of the trough toward the main gullet. Level inverts, however, are generally used with the average filter. Under these circumstances, the width of the trough would be selected as a reasonable value, and the depth of the trough would be computed by the following formula:

$$\text{Depth of trough} = 2/3 \frac{Q}{2.49 \times W}, \text{ where}$$

Q - cfs of wash water entering each trough, determined from the wash water rising throughout the area tributary to each trough.

W - assumed width of trough.

For example, assume a trough width of 1.25 feet for a trough 14 ft. long in a filter 14.5 x 14 ft. in size. Then two troughs 6 ft. apart and 3 ft. from each well would suffice. Assume a wash water rate of 16 gpm/sq. ft. Then each trough would receive 16 gpm x 7.25 x 14 = 1674 gpm = $\frac{1624}{60}$ gps = 27.0 gps = 4.52 cfs.

$$\text{Depth of trough} = 2/3 \left(\frac{4.52}{2.49 \times 1.25}\right) = 2/3\ (1.45) = 1.28 \text{ or } 1.3 \text{ ft.}$$

The height of the upper edge of the trough should be the trough depth, 1.3 ft., plus the thickness of the concrete of the trough, 0.2 ft., plus 1.3 ft. for sand expansion, plus 0.2 ft. factor of safety, or a total of 3.0 ft.

An independent approach to this problem is to use the empirical value for the height of the trough edge above the sand as equal to the distance the water rises in one minute. At 20.0 gpm/sq. ft., this is 2.7 ft., a good check of the computed value. This height of the wash water trough of 3.0 ft. will provide freeboard for washing with water of lower temperature than 18° C, or will permit washing at rates greater than 20 gpm/sq. ft. with warm waters.

Rate Controllers - The rate of filtration must be controlled automatically so that it will be uniform in spite of the gradual increase in the loss of head as the sand becomes more and more impervious during the filtering cycle. This is most conveniently accomplished by adjustable rate controllers, which incorporate their own metering device and which maintain the selected rate independent of the depth of water on the sand and the prevailing loss-of-head. These usually are fitted with rate indicators and even recorders. Such complete units, however, are expensive, and are difficult to repair. Therefore, their use usually is restricted to larger plants where budget and staff are adequate for proper maintenance and operation.

Float-operated butterfly valves provide a satisfactory substitute, because of their simplicity and relatively low cost. These valves compensate for the varying loss-of-head during the filtering cycle, but they do not maintain a constant rate of filtration unless the rate of flow of water to the filter is controlled. Therefore, a float-controlled butterfly valve, with the float in a Parshall flume must be used on the raw water piping, unless reasonable constant capacity raw water pumps are to be used, or where pump capacity is measured by the flume and is subject to manual control by a butterfly valve. Furthermore the equal distribution of settled water to each filter, irrespective of the elevation of the water surface in each filter, must be provided by using weirs on the filter influent piping.

Appendix to this Bulletin gives an example of a design using butterfly valves with adjustable floats.

Loss-of-Head Indicators - Loss-of-head indicators are needed to indicate the condition of the filter sand and when a filter must be washed. They are a part of elaborate rate controllers and rate of flow indicators used in larger plant. A simple gauge for small plants consists of two glass tubes on each side of a calibrated scale, with one tube connected to the effluent pipe between the filter and rate-controller, and the other connected to the filter structure above the sand. The relative elevation of the water surfaces in these tubes indicates the prevailing hydraulic gradient, or loss-of-head through the filter.

The operation of filters is facilitated by the use of hydraulically operated valves. These, however, are expensive and require a separate high pressure water system. Therefore, manually operated sluice gates and plug valves generally are preferable for plants where large-size valves are not needed. Quick opening plug valves are available for wash water drains from 8 to 12 inches in size.

Filter-to-waste valves and piping are not needed, as any water so wasted comes from the pores in the gravel and sand, and hence represents previously filtered water and not the water first passing through the upper portion of the sand when the filter is placed in operation after washing.

In general filtered water and wash water piping should be designed to provide a velocity of flow not greater than 3 to 6 and 8 to 12 fps respectively.

The pipe gallery should be ventilated so as to reduce humidity, and adequate illumination should be provided for maintenance and repairs. Walkways over piping are needed when filters are located on both sides of a gallery.

CHAPTER VIII. CHLORINATION

Objectives - Public water supplies should be disinfected to remove bacterial pollution found present in otherwise satisfactory ground waters or as an adjunct to the filtration or other treatment of surface waters. Chlorination is the established, practical and economical process for the disinfection of public water supplies, and thus is the only process considered herewith. Chlorination should not be considered as a substitute for good design, sound construction or proper operation of ground water supplies, inasmuch as the quality of water from properly constructed, located and maintained wells is more certain and reliable than that of a polluted raw water supply after chlorination.

The limitations of treatment by chlorination alone should be considered, so that undue reliance will not be placed in the disinfecting process when other types of treatment also are required. Generally speaking, treatment by chlorination without filtration, is effective and adequate under the following conditions:

a. the degree of bacteriological pollution is moderate and reasonably uniform, and the bacteria being destroyed are not shielded from the chlorine by being bedded in suspended solids;

b. the turbidity and color of the water do not exceed 5 to 10 units;

c. the iron and/or manganese content of the water does not exceed 0.3 ppm;

d. the chlorine demand of the water does not fluctuate so rapidly as to prevent proper adjustment of the chlorine dose;

e. taste and odor producing substances are absent or do not interfere with the selection of adequate chlorine doses through the production of chlorine tastes; and

f. there is a contact period of at least 15 minutes between the point of chlorination and the house connection of the consumer first supplied with water. More effective results can be secured with doses of chlorine sufficient to oxidize organic matter, ammonia, etc., and produce "free residual chlorine," rather than chloramines or "combined residual chlorine".

Consumers in many countries demand clear attractive water of safe sanitary quality. The above conditions, therefore, pertain to factors affecting treatment by chlorination alone.

Chlorine is an active oxidizing agent and thus should be appraised as an adjunct to iron and manganese removal. It also is a bleach and thus aids in color removal. The cost of chlorination for these purposes, however, should be considered in the light of the chlorine demand of the specific water supply, before being adopted. Pre-chlorination before filtration is not always economically feasible, because of the higher chlorine demand of many raw waters, as compared to filtered waters, especially where the cost of chlorine is high. There are instances, however, where sewage-polluted streams must be used as sources of water supply and where treatment by pre-chlorination, coagulation, sedimentation, filtration and post-chlorination will produce a water of attractive characteristics and of safe sanitary quality. Even this elaborate treatment procedure has limitations, which should be compensated for by stream pollution control, storage and presedimentation, or by special taste and odor control procedures, which may not be considered feasible, because of the cost of chlorine in high oxidation doses, or other chemicals which must be imported.

The needed degree of treatment basically is related to the degree of bacteriological pollution, that is the most probable number (M.P.N.) of coliform organisms per 100 ml. in a series of representative samples collected over a period sufficiently long to show the range in fluctuations of quality. The U.S.P.H.S. after prolonged research has concluded that treatment by coagulation, sedimentation, filtration and post-chlorination is adequate when the monthly average M.P.N. value is 5,000 per 100 ml. or less, with this value being exceeded in less than 20% of the samples. Pre-chlorination also should be practiced when the monthly average M.P.N. is 5,000 per 100 ml. or less and when the M.P.N. exceeds 5,000 per 100 ml. in more than 20% of the samples, but does not exceed 20,000 per 100 ml. in more than 5% of the samples. Any higher degree of pollution of raw waters is undesirable, but if local conditions warrant and an effective operation and laboratory control is assured, then more effective pre-chlorination should be provided by using doses of chlorine sufficient to produce free residual chlorine over the long period required for the water to pass through flocculation and sedimentation basins and the filters, with at least 0.2 ppm free residual in the filter effluent prior to post-chlorination. The post-chlorine dose then becomes a factor of safety, sufficient to increase the residual to say 0.4 ppm.

Chloramination, or chlorine-ammonia treatment, has become obsolete because the stability of residual chlorine secured through the reaction between chlorine and ammonia has been found to be at the expense of rapidity and effectiveness of disinfection. Furthermore the cost of ammonia exceeds the cost of any needed increase in chlorine dose to secure free residual chlorine.

Chlorine Chemicals - Liquefied gaseous chlorine, sodium hypochlorite, calcium hypochlorite and chloride of lime are all acceptable for the treatment of water. The availability, shipping charges, costs, ease of handling and application should be considered in the selection of the chlorinating agent to be used for a specific supply. Ordinarily the use of liquefied gaseous chlorine is justified only for the large supplies served by established and economical shipping facilities, because of the cost and complexity of gas chlorinators. The stability during storage, the greater solubility and the higher chlorine content of high test calcium hypochlorite makes it preferable to chloride of lime. It is sold under several trade names of "H.T.H.", "Perchloron", "Pittchlor", etc.

The properties of liquefied chlorine gas, and of chlorine compounds are shown in Table II.

Storage - Storage facilities should be provided for an adequate supply of chlorine or chlorine compounds in the light of availability of supply and shipping facilities. Cylinders of liquid chlorine preferably should be placed in a separate room with an outside entrance, the door being

TABLE II

"PROPERTIES OF CHLORINE DISINFECTANTS"

CHARACTERISTICS AND RECOMMENDATIONS	CALCIUM HYPOCHLORITE	CHLORINATED LIME	CHLORINE	SODIUM HYPOCHLORITE
FORMULA	$Ca(OCl)_2 \cdot 4H_2O$	$CaO \cdot 2CaOCl_2 \cdot 3H_2O$	Cl_2	NaOCl
COMMON NAME	(H.T.H., Perchloron, Pittchlor)	Variable formula-(Chloride of Lime, Bleaching powder)	(Chlorine Gas, Liquid Chlorine)	(Javelle Water, Bleach Liquor, chlorine bleach)
USE	Disinfection, slime control, Deodorization	Disinfection, Slime Control	Disinfection, Slime Control, Taste and Odor Control, Waste Treatment, Activation of Silica (See Footnote 5)	Disinfection, Slime Control
AVAILABLE FORMS	Granules, Powder, Tablets	Powder	Liquefied gas under pressure	Solution
CONTAINERS AND REQUIREMENTS	Bbl-415 lb Cans-5, 15, 100, 300 lb Drums-800 lb	Drums - 100, 300, 800 lb	Steel cylinders 100, 150 lb. Ton Containers T/C-15 ton containers T/C-16, 30, 55 tons Green Label	Carboys-5, 13 U.S.gal Drums-30 gal Bulk-1300, 1800, 2000 USgal T/C
APPEARANCE AND PROPERTIES	White, non-hygroscopic, corrosive, strong chlorine odor	White, unstable, deteriorates, chlorine odor Tends to precipitate incrustants in hard water in feeders	Greenish yellow gas liquified under pressure Pungent, noxious, corrosive gas heavier than air; health hazard	Yellow liquid, strongly alkaline Store in cool place, protect from light and vent containers at intervals
WEIGHT lb/cu ft (Bulk Density)	50 to 55	45 to 50	Spec.Grav. with respect to air 2.49	————
COMMERCIAL STRENGTH	70% Available Cl_2	25 to 37% Available Cl_2	99.8% Cl_2	13.0% NaOCl or 12.5% Available Chlorine
SOLUBILITY IN WATER gm/100 ml	21.88 ( ) 0°C 22.7 ( ) 20°C 23.4 ( ) 40°C	Decomposes to form hypochlorous acid	1.46 ( ) 0°C 0.98 ( ) 10°C 0.716 ( ) 20°C 0.57 ( ) 30°C	Completely miscible
BEST FEEDING FORM	Up to 2% Soln. as available Cl_2-Max.	2% soln. of Avail. Cl_2 - Max.	Gas - vaporized from liquid	Solution Up to 12.5% Avail. [illegible] NaOCl soln. may be made by adding Na_2CO_3 to $Ca(OCl)_2$ soln.
CHEMICAL TO WATER RATIO FOR CONTINUOUS DISSOLVING	0.15 lb/gal makes 1% soln. of available Cl_2	About 0.3 lb/gal makes 1% soln. of available Cl_2	1 lb. to 45-50 gal or more	1.0 US gal of 12.5% (Avail. Cl_2) soln. to 15.7 US gal of water gives [illegible] available Cl_2 soln.
TYPES OF FEEDERS	LIQUID Prop. Pump - up to 45 gph at 100 psi Precision-up to 25 gph at gravity feed	LIQUID Pro. Pump-Up to 45 gph at press. up to 100 psi Precision-Up to 25 gph at gravity feed	Gas Chlorinizers	SOLUTION Prop.pump-up to 45 gph at press. up to 100 psi Precision-Up to 25 gph gravity feed Sampler Pump-Up to 5 gph, at [illegible] psi
ACCESSORY EQUIPMENT REQUIRED	Dissolving tanks in pairs with drains to draw off sediment	Dissolving tanks in pairs with drains to remove sludge	Vaporizers for hight capacities	Solution tanks, Foot valves Water meters, Injection nozzles
SUITABLE HANDLING	Ceramic, glass, plastics or rubber lined tanks No tin	Rubber, stoneware, glass, plastic lined tanks No tin	DRY LIQUID or GAS Black iron, copper steel WET GAS - Glass, Silver, Hard, rubber tantalum	Rubber, plastic, glass, ceramics

fitted with a ventilating opening at the floor level, so that any leakage of gas, which is heavier than air, will be to the exterior. An exhaust fan provides an additional safety factor at large installations. Facilities should be provided to connect two or more 150 pound cylinders of chlorine to the chlorinator, so that not more than 35 pounds of chlorine will be withdrawn from one cylinder in 24 hours, which is the limit in the rate of evaporation of liquefied chlorine, from small cylinders.

The storage space for drums of dry chloride of lime or calcium hypochlorite, or large bottles or plastic drums of sodium hypochlorite should be as dry and as cool as is available. The storage space for bottles of sodium hypochlorite should be enclosed to exclude light and thus decrease the rate of loss of chlorine. Scales should be provided for weighing chloride of lime or calcium hypochlorite or on which to place cylinders of liquid chlorine.

Sodium and calcium hypochlorite solutions are very corrosive. Sodium hypochlorite is best applied as received, with a strength of about 15% chlorine. If 15% solution must be diluted to permit the accurate chlorination of small supplies, such dilution should be made from day to day as needed, because the more alkaline solution as purchased is much more stable than it is after dilution, and hence should be stored undiluted.

Chloride of lime contains an excess of insoluble lime, which does not dissolve. Therefore, solution tanks should have an ample drain and sludge storage space, and the outlet hose leading to the chlorinator should be elevated about 6 inches above the bottom of the solution tank, so that the settled sludge does not enter the outlet hose.

Concrete solution tanks should be coated on the inside with bitumastic enamel.

<u>Chlorinators</u> - Chlorinators should be in duplicate and spare parts should be available to facilitate repairs.

Chlorinators for feeding liquefied, gaseous chlorine should be located as mentioned above for chlorine cylinders.

Dry-feed, gas chlorinators should be used only when water under pressure is not available, or when power is not available to operate a small pump, or when local conditions are not favorable to the operation of hydraulic pumps, otherwise needed with solution-feed chlorinators. Solution-feed, gas chlorinators should be supplied with water under a pressure of at least three times the pressure of the water supply at the point of treatment, and at the volume required to operate the injector of the specific chlorinator selected. Chlorine preferably should be introduced into the suction pipes of pumps when feasible.

Any small pumps required for the operation of chlorine injectors of solution-feed chlorinators against pressure should be driven by the main engines or by electric motors connected to a reliable source of current.

In general the convenience and economy of using liquefied, gaseous chlorine is counterbalanced by cost and complexity of gas chlorinators. Assuming a useful life of 10 years for a chlorinator, the difference in cost of a gas-feeding unit versus a hypochlorinator, may be compared with the difference in cost of calcium hypochlorite versus liquid chlorine, to determine at what point it is more economical to use the more costly gas feeding unit.

Hypochlorinators may consist of: small motor-driven, diaphragm pumps; meter-paced, water operated pumps providing automatic compensation of dose with rate of flow; constant level, calibrated orifice feeders; or variable head, orifice feeders; each with capacities between a fraction of a gill per minute to many gallons per day.

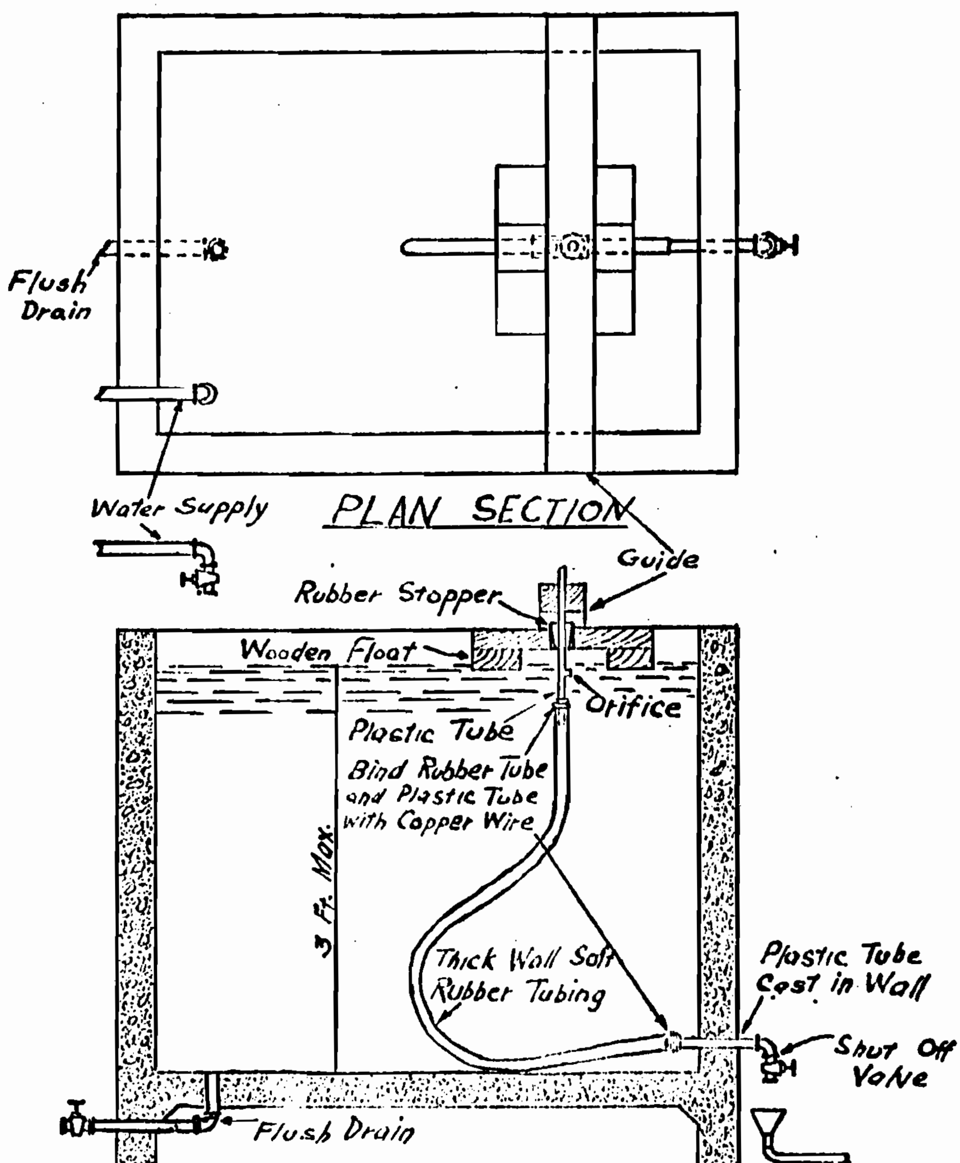
Hypochlorinators driven by electric motors and subject to manual control should have their motors wired to the main switchboard so the units will be started and stopped in coordination with the main pumps. Motor-driven hypochlorinators preferably should not be used when electric power is unreliable, or when a gravity supply is being treated, so as to avoid interruption in treatment of the water which will continue to flow to the distribution system, even when electric power fails. Water operated units, either manually or meter controlled, are available for pressure ranges of 10 to 125 pounds.

A home-made unit, capable of close control and low rate of feed, consists of a 10 gallon bottle, together with the following: The stopper of the bottle is replaced by a rubber stopper of the same size perforated with two holes. A glass tube is passed through one of the holes of the stopper to within 4 inches of the bottom of the bottle, this tube being fitted at its upper end with a short length of rubber tubing with a laboratory screw-type, pinch clamp. A second glass tube is passed through the stopper and extends to the bottom of the bottle. Its upper end is bent in two 90° bends, and a rubber tube is attached, extending outside the bottle to a point a few inches below the bottom of the bottle.

The principle of operation is as follows:

The pinch clamp on the tube at the top of the straight glass tube controls the entrance of the air into the bottle. The elevation of the lower end of this same glass tube controls the net head operating the siphon formed by the second glass tube and attached rubber tubing. The rate of flow may be controlled, therefore, by adjusting the net head and also by controlling the rate of admission of air. This latter is the procedure followed, because it is less convenient to raise and lower the elevation of the tube. The unit is calibrated to establish the head required to give the maximum rate of flow of solution needed for any given supply, and thereafter the entrance of air is adjusted as necessary.

A large-capacity, home-made unit, for preparing and also feeding hypochlorite solution is shown by Figure 5. The concrete tank may be given any desired capacity, or a wooden barrel may be used. For instance, a tank with an effective size of 4 x 4 x 4 ft., filled with 400 gallons of water in



ELEVATION SECTION

EMERGENCY EQUIPMENT FOR FEEDING HYPOCHLORITE SOLUTION

FIGURE 5.

which 57.2 pounds of 70% strength calcium hypochlorite is dissolved would give 40 pounds of chlorine in the form of a 1% solution. This solution would chlorinate 4 million gallons of water with a dose of 1.0 ppm, or would provide a reserve capacity for a maximum dose of 3.0 ppm for 1.33 million gallons of water.

The inner surface of the concrete tank should be coated with bitumastic enamel or plastic. The drain should be hard rubber or plastic tubing with rubber stopcock. Tubing supported by the float should be plastic, or hard rubber, although copper or brass pipe will give reasonable life. The flexible tubing leading to the outlet pipe should be plastic or rubber hose. The float may be enamel coated wood.

An orifice is drilled in the side of the upright tube, which must be open at its upper end. An orifice about 1/2" below the surface of the solution and 1/8" in diameter will have a capacity of 48 gallons per day. A head of 4-1/8 inches would give a flow of slightly less than 400 gallons per day. The actual flow may be calibrated by timing the period required to fill a liquid measure held above the funnel. The flow would be adjusted by changing the weight on the float or by adjusting the position of the float, thus altering the head on the orifice as desired.

The solution hose from the funnel cannot be connected directly to the suction pipe of a pump, unless a water-seal tank is used to prevent the entrance of air.

Point of Application - The point of application of chlorine should be selected to give the maximum possible contact period for the chlorine before the treated water reaches the first consumer. The minimum period preferably should be 30 minutes and even longer when the pH of the water exceeds 7.6, or when the water is subject to sewage pollution. It is best to provide baffles in filtered water basins to prevent short circuiting from inlet to outlet, and to apply chlorine at the inlet, and any lime used for corrosion prevention to be applied near the outlet, so that pH is not increased before the chlorine has had considerable time to react at lower pH values.

Summary - The effective and economical chlorination of a specific water supply requires the consideration of:

a. the quality and chlorine demands of the water to be chlorinated, and the range in required dose of chlorine;

b. the selection of one of the chlorine chemical available locally or which may be transported and stored economically;

c. the selection of a chlorinator suited to specific local conditions such as hydraulic factors, and which is sold by a local firm maintaining spare parts and service facilities;

d. the selection of a point of chlorination which provides effective mixing of the chlorine in the water and the maximum available period of contact; and

e. the overall appraisal of the proposed facilities in the light of the reliability of the operator and funds available for operation.

Manually controlled chlorinators are considered to be satisfactory for the treatment of pumped supplies, or gravity supplies flowing at rates which do not vary more than 20% from minimum to maximum values. When the flow varies over a range of more than 10% ±, an automatic, proportional-feed chlorinator should be used, the meters or other control devices being selected with due regard to the range in the fluctuations in flow and other hydraulic factors. The range in flow may be greater than the range in capacity of metering devices or a single chlorinator, namely 1 to 10, in which case compound meters or their equivalent should be used to control one or two chlorinators selected to have the required total range in capacity. For instance, the maximum flow during the day, plus the water used in fighting fires, may be well over 10 times the minimum night flow.

The maximum capacity of chlorinators should be selected with due regard to the maximum amount of water to be treated and the anticipated maximum chlorine demand of the water, as determined by the dose sufficient to produce a concentration of residual chlorine of at least 0.5 p.p.m. after at least 30 minutes contact period. Higher capacity will be needed when "free residual chlorination" is to be practiced for more rapid disinfection or for taste and odor control. A maximum capacity sufficient for a dose of 5.0 p.p.m. is desirable.

CHAPTER IX. IRON AND MANGANESE REMOVAL PLANTS

This chapter deals with special iron and manganese removal processes, and not with conventional filtration plants, which may serve to remove these minerals when present in turbid or colored surface waters. Ordinarily iron and manganese present a problem only with well waters, or surface waters stored in deep reservoirs.

The objective should be to deliver water to the public sufficiently free of iron and manganese to prevent the staining of laundry and plumbing fixtures. The maximum permissible concentration depends upon the chemical state of the minerals, that is upon whether the minerals remain in solution as invisible, stable compounds, or whether they precipitate readily upon oxidation in the distribution system or during the use of the water, especially when alkaline soap is used in washing clothes. If these minerals are in the easily oxidizable state, experience has shown the maximum desirable concentration in drinking waters to be 0.3 ppm. for either mineral alone or in combination. Somewhat higher concentrations of soluble iron or manganese are acceptable.

The literature as to iron and manganese removal processes is confusing. Therefore, a tabulation has been added to this chapter summarizing the significant aspects of ten processes.

The basic problems of design are related to the characteristics of the specific water supply and the chemical composition of the iron and manganese compounds in the supply under study. Ferrous bicarbonate alone usually may be precipitated upon aeration without lime treatment, when the removal of carbon dioxide by aeration raises the pH to about 7.0 or more. Ferrous bicarbonate also may be precipitated as ferrous carbonate by lime treatment to produce a pH of 8.1, provided oxidation is prevented by excluding air. This eliminates the need for aeration (See process 10). If, however, iron is combined with organic matter in soluble, stable compounds, lime treatment, following aeration, may be needed to produce a pH of 8.5 or more, unless catalytic action in contact beds, or chlorination results in oxidation and precipitation at lower pH values.

The removal of manganese is more complicated than iron removal. Oxidation resulting from catalytic action may require a pH of at least 8.5, and pH of at least 10.0 with treatment by aeration, lime application, sedimentation and filtration. However, manganese compounds may be changed to manganous hydroxide, in the absence of air, when closed contact beds of the manganese ore, pyrolusite, are used. The soluble manganous hydroxide remaining in the water then is oxidized to insoluble manganic hydroxide by aeration through a ventilated contact bed or aerator, and the water is then filtered.

If iron is present with a limited amount of manganese, it may be feasible and economical to remove the iron but not the manganese, which would remain as stable, soluble compounds in the effluent. Soluble manganese may have no staining characteristics even in concentrations of 1.0 ppm.

The water under study, therefore, should be tested to determine whether iron or manganese are present alone or whether both minerals are involved. Washing white cotton cloth in the water with laundry soap will disclose the significance of these minerals. Tests should be made whether aeration without lime treatment results in precipitation, or whether lime treatment or contact beds are needed to secure oxidation and precipitation. If the latter is the case, usually a small pilot plant is needed to establish the best and most economical design for the plant under study.

SUMMARY OF IRON AND MANGANESE REMOVAL PROCESSES

The physical and chemical changes and reactions occurring in iron and manganese removal, therefore, are complex. The table at the end of this chapter is intended to be illustrative.

Engineering study should be directed to establishing whether the pH, carbon dioxide and organic matter in the specific water stabilizing the iron or manganese, that is whether water characteristics are such that the oxidation and precipitation of these minerals cannot be secured by simple aeration without pH adjustment or catalytic action. In general waters will be found to fall into five types, as follows:

1. Waters containing only iron, and with a pH of 7.0 or more after aeration, which will oxidize upon aeration, without further pH adjustment or catalytic action being required.

2. Waters containing iron and also a limited amount of manganese, where oxidation requires catalytic action in contact beds or chlorination or pH adjustment prior to filtration.

3. Waters containing iron bound with organic matter and where pH adjustment to values of 8.5 to 9.0 is required to secure oxidation and precipitation, or waters containing manganese which may not oxidize at pH values below 10.0.

4. Waters having a high hardness and where iron and/or manganese removal results from lime-soda, or zeolite softening.

5. Waters containing manganese alone or with iron, and where the catalytic action of pyrolusite ore is used in the absence of air to change complex manganese compounds to manganous hydroxide. (See process No. 5). The manganous hydroxide is oxidized, together with any iron which may be present, by aeration in a secondary contact bed followed by filtration. No lime treatment is needed for pH correction, provided the pH is 7.0 or higher. Note should be made that preliminary aeration must be avoided, otherwise manganous hydroxide will not be formed in the contact bed, but a colloidal form of manganese will be formed, which precipitates very slowly, unless lime or chlorine are added, as in processes No. 4 or No. 6.

Design Details - The various units of iron and manganese removal plants are interrelated and hence must be coordinated in the design. The respective structures needed for specific phases of iron and manganese processes are discussed below.

Aerators - Aerators are usually intended to remove carbon dioxide and taste and odor producing substances, and to introduce oxygen, as discussed in Chapter IV. Spray type aerators are most effective for these purposes, but iron and manganese removal is facilitated when the water is aerated while trickling down through beds of gravel, the coating on which acts as primary contact beds for catalytic action. The gravel is placed on 4 to 6 trays in layers about 12 inches deep, the trays being separated by air spaces and perforated to distribute the falling water. Single beds 4 to 6 feet deep are satisfactory, provided ventilation is insured by perforation in the retaining walls. Louver-like construction of retaining walls is most satisfactory in providing ventilation and facilitating the cleansing of the gravel bed with a hose stream. Rates of flow of 10 gpm/ sq. ft. are satisfactory.

Contact Beds - The purpose of contact beds is to facilitate oxidation of iron or manganese through the catalytic action of previously precipitated oxides of these minerals on the gravel or ore. Superior results are claimed for the manganese ore, pyrolusite, which is an oxide of manganese. Usually preference is given to upward flow at rates up to 4.0 gpm/ sq. ft. Bed depth should be 6 feet, or any greater depth (or lower unit rate of flow) found necessary by pilot plant studies. Provisions must be made for the rapid draining of the beds so as to wash excess oxides from the gravel or ore. Provisions also should be made for the use of a hose stream for periodic cleansing the gravel or ore.

Contact beds of pyrolusite ore, for manganese removal without lime treatment, (see process 5 in appendix of this Chapter) must be in closed structures to prevent the entrance of air. Upward flow, at rates established by pilot plant tests, should be provided. A trial rate of 2.0 gpm/ sq. ft. with a bed depth of 6 feet is suggested, giving a contact period of 9 minutes, with usual void volume of 40%. The effluent from such beds should be aerated in a downward flow, contact bed type of aerator constructed to facilitate passage of air, as discussed above. Final filtration is needed as discussed below.

Lime Treatment - Equipment for feeding lime for pH correction should be selected with due consideration to the factors discussed in Chapter VI. The capacity of the lime feeder should be at least 50% greater than dose of lime found to be needed to raise the pH of the water to the point where laboratory studies with the specific water show the iron and or manganese to be oxidized and coagulated.

Oxidation by Chlorination - Equipment for this purpose should be selected with due consideration of the factors discussed in Chapter VIII. Consideration should be given, before adopting this process, as to whether chlorine, in the absence of dissolved oxygen, is effective in oxidizing iron or manganese in the specific water supply under design, and whether the cost of chlorination equipment and chlorine is less than the cost of aeration and double pumping otherwise required. This process is especially adaptable to those instances where disinfection of a well supply also is needed, or where oxidation of iron is secured thereby without lime treatment for pH adjustment. Manganese is oxidized by "free" residual chlorine, but the reaction rate is slow, unless the pH exceeds 8.5 to 10.0.

<u>Sedimentation Basins</u> - The engineering design should establish the need for sedimentation following aeration, catalytic action in contact beds, or chemical treatment prior to filtration. More economical filtration usually is secured with sedimentation, when lime treatment is needed, or when the iron content of the raw water exceeds about 2.0 ppm. Refer to Chapter VII for features of design or sedimentation basins. Sedimentation periods should be established for a specific water, but usually may be from 1 to 2 hours for iron removal.

A plant designed to remove iron <u>in the absence of oxygen</u>, in accordance with process 10 of the tabulation in this chapter, requires flocculation, sedimentation and filtration <u>under pressure</u> to avoid exposing a well water to the atmosphere. Consideration should be given to the use of a cylindrical flocculation tank of a size providing about 5 minutes detention and with inlet and outlet so located as to produce spiral flow to provide agitation. Sedimentation under pressure may be secured in a larger cylindrical tank providing a minimum detention period of 1 hour.

<u>Filters</u> - Filters used to remove the iron or manganese remaining in the effluents of aerators, contact beds or sedimentation basins should be designed as discussed in Chapter VIII. Rates of filtration of 4.0 gpm/sq. ft. or less are satisfactory. The filter sand should have an effective size of 0.7 to 1.0 mm, that is coarser than with filters intended for bacterial removal. Open, gravity filters, and also pressure units may be used, as the requirements are not as rigid as with conventional filters used for bacterial removal.

Filters may serve adequately as contact beds following aeration, provided iron alone is involved. This should be established for a specific supply by pilot plant studies, because of the economy of eliminating a separate contact bed.

<u>Ion-Exchange Softeners for Iron and Manganese Removal</u> - Conventional ion-exchange softeners, discussed in Chapter XI, should be considered for hard waters containing iron and/or manganese, provided the raw well water is devoid of oxygen, inasmuch as the ion-exchange process removes only <u>soluble</u> iron and manganese, together with the calcium and magnesium. This process should not be used when the iron and manganese content exceeds 0.5 ppm for each 17.0 ppm of hardness, up to a maximum of 10.0 ppm. As aeration or pH adjustment is not needed the cost of double-pumping is avoided.

PROCESSES OF IRON AND MANGANESE REMOVAL

Treatment Processes	Oxidation Required	Character of Water	Equipment Required	pH Range Required	Chemicals Required	Remarks
1 Aeration Sedimentation Sand filtration	Yes	Iron alone in absence of appreciable concentrations of organic matter	Aeration, settling basin, sand filter	Over 7.0	None	Easily operated. No chemical control required.
2 Aeration, contact oxidation, sedimentation, sand filtration	Yes	Iron and manganese loosely bound to organic matter, but no excessive carbon dioxide or organic acids content	Contact aerator of coke, gravel or crushed pyrolusite, settling basin and sand filter	Over 7.0 for iron removal. Over 10.0 for manganese removal	None	Double pumping required. Easily controlled.
3. Aeration, contact filtration	Yes	Iron and manganese bound to organic matter, but no excessive organic acid content	Aerator and filter bed of manganese coated sand, "Birm," crushed pyrolusite ore, or manganese zeolite	Over 7.0 for iron removal. Over 10.0 for manganese removal	None	Double pumping required unless air compressor, or "sniffler valve" is used to force air into water. Limited air supply adequate. Easily controlled.
4 Contact filtration	Yes, but not by aeration	Iron and manganese bound to organic matter, but no excessive carbon dioxide or organic acid content	Filter bed of manganese coated sand, "Birm", crushed pyrolusite ore, or manganese zeolite	Over 7.0 for iron removal Over 8.5 for manganese removal	Filter bed reactivated or oxidized at intervals with chlorine or sodium permanganate	Single pumping. Aeration not required.
5 Catalytic action, aeration, sedimentation and filtration.	Yes	Manganese in combination with organic matter	Closed pyrolusite bed, aerator, second open contact bed, sand filter -	Over 7.0	None	Manganese changed to manganous hydroxide by catalytic action in absence of air, and then oxidized -
6 Aeration, chlorination, sedimentation and sand filtration	Yes	Iron and manganese loosely bound to organic matter	Aerator and chlorinator or chlorinator alone, settling basin and sand filter	7.0 to 8.0	Chlorine	Required chlorine dose reduced by previous aeration but chlorination alone permits single pumping
7 Aeration, lime treatment, sedimentation, sand filtration	Yes	Iron and manganese in combination with organic matter, and organic acids -	Effective aerator, lime feeder mixing basin, settling basin sand filter	8.5 to 10.0	Lime	pH control required
8 Aeration, coagulation and lime treatment, sedimentation sand filtration	Yes	Colored, turbid, surface water containing iron and manganese combined with organic matter	Conventional rapid sand filtration plant -	8.5 to 9.6	Lime and ferric chloride or ferric sulphate, or chlorinated copperas, or lime and copperas	Complete laboratory control required -
9 Zeolite softening	No	Well water devoid of oxygen, and containing less than 0.5 p.p.m. iron and manganese for each 17.0 p.p.m. hardness removed	Conventional sodium zeolite unit, with manganese zeolite unit or equivalent for treatment of by-passed water	Over 6.5+	None added continuously, but bed is regenerated at intervals with salt solution	Only soluble ferrous and manganous bicarbonate can be removed by base exchange, so aeration or double pumping is not required
10 Lime treatment, sedimentation, sand filtration	No	Soft well water devoid of oxygen containing iron as ferrous bicarbonate	Lime feeder, enclosed mixing and settling tanks and pressure filter	8.1 to 8.5	Lime	Precipitation of iron in absence of oxygen, as ferrous carbonate. Absence of oxygen minimizes or prevents corrosion. Double pumping not required.

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CHAPTER X. SOFTENING

Objectives - The demand of the public for soft water is influenced by the individual's experience with waters familiar to them. Those familiar with soft waters will object to a hardness over about 50.0 p.p.m., whereas others will be pleased with waters with a hardness of 100.0 p.p.m. Generally softening is economically justified when the hardness exceeds about 150.0 p.p.m., but engineering studies of many local factors, including cost of chemicals, availability of competent operating personnel and funds, should be made to establish the need and practicability of softening any given supply.

Industries needing waters of very low hardness should install their own softening equipment, because it would be uneconomical to soften the whole public water supply by the lime-soda process to the fullest theoretical degree, and complete softening by the base exchange process also would be expensive and in addition would produce a corrosive water.

General - The details of the lime-soda and base exchange softening processes should be considered for each specific design in the light of local conditions. In general the softening of a surface supply usually justifies the selection of the lime-soda process, so that clarification also can be provided, whereas clear well waters usually can be softened more economically by the base exchange process. The latter process also is more easily controlled and operated by those lacking technical training. Furthermore, the design of base exchange units has become highly standardized and complete units may be purchased.

LIME-SODA WATER SOFTENING PLANTS

Unless otherwise discussed in this chapter, the details of design of such plants are similar to those of rapid-sand filtration plants, altered so as to more adequately meet the requirements of the lime-soda softening processes. Therefore, the following discussion will be restricted to the specific features or altered aspects of softening plants.

Softening Processes - The designer should give consideration to the several softening processes so as to select the best for the softening of a specific water under prevailing load conditions and limitations, and especially the availability of laboratory control of the proposed plant. In general there are three variations in the softening process, which should be considered.

1. Excess lime treatment - An excess of lime (53 to 75 p.p.m.) is added to precipitate magnesium and calcium at pH 10.6. Soda ash then is added to convert the excess of lime to insoluble calcium carbonate and soluble caustic soda. The caustic soda, that is caustic alkalinity, is undesirable, so the second process is prefered.

2. Split lime treatment - A major portion of the raw water is given the excess lime treatment to precipitate magnesium at pH 10.6 in a primary basin. The remainder of the raw water is added, the carbon dioxide and bicarbonate content of which reacts with the excess of lime to precipitate the calcium at pH of about 9.4 in a secondary basin.

3. Excess lime treatment with recarbonation - Excess of lime is added to precipitate magnesium at pH 10.6 and the water is flocculated and settled. Carbon dioxide gas then is added to reduce the pH to 9.4 favorable to changing bicarbonate to the insoluble calcium carbonate. Soda ash is added as needed to precipitate the permanent hardness, present in the form of calcium sulphate and chloride, during secondary flocculation and sedimentation. Carbon dioxide is added a second time to reduce the pH to about 8.7 to stabilize the water and reduce incrusting the filter sand by changing the residual carbonate back to soluble bicarbonate.

Comment - The treatment is simplified when magnesium hardness is not present. The trend is to use the second process to avoid the cost and complexity of recarbonation of process 3. More recently another trend is to return a portion of the sludge to the raw water to provide the advantages of "solid contact" flocculation, otherwise secured in "solid contact basins". If the quantity of returned sludge is such as to maintain a sludge slurry of about 3% on a dry weight basis, flocculation is greatly aided and quickened.

Chemical Feeders - The large doses of lime and soda ash required for this process usually justify the use of dry chemical feeders. (Lime dose of 300 p.p.m. is required to reduce hardness from 300 to 100 p.p.m.) The available supply and costs of hydrated lime and quick lime should be considered. Lime slaking equipment of the continuous flow type should be provided if quick lime is selected for large plants. Three feeders should be provided, one of which should be adapted to feeding either lime or soda ash, so as to serve as a reserve unit. This spare unit also will be available for feeding alum, if coagulation is found to be desirable.

Chemical Storage and Handling Equipment - These facilities and equipment should be similar to those discussed in Chapter VI, except that allowance must be made for storing, handling and feeding larger quantities of chemicals.

Flocculation Basins - Prolonged agitation for periods of at least 20 to 40 minutes should be provided to insure the completion of the softening reactions and flocculation before the water enters the sedimentation basin. Mechanical flocculation equipment usually is justified with the softening process, because of the critical importance of the flocculation and precipitation and the large amount of soluble material being precipitated.

Sedimentation Basins - Such basins should be designed with due consideration to the factors discussed in Chapter VII. The large amount of sludge justifies the consideration of the use of mechanical sludge removing equipment at large plants, which also permits the return of a portion of the sludge to the raw water.

Sludge Disposal - Inasmuch as about 2.0 p.p.m. dry solids are produced for each p.p.m. hardness removed, consideration should be given to the disposal of sludge so as not to create a nuisance or seriously pollute the receiving stream. The use of settling lagoons should be considered when the receiving stream is small and only able to handle the supernatant water flowing from such lagoons. A minimum depth for such lagoons of about 4 feet is desirable.

Stabilization - Unless "split treatment" is practiced, provisions should be made to stabilize the softened, settled water prior to filtration to prevent disposition on the filter sand and in the distribution system and to prevent tastes in the treated water due to the caustic reaction. The hardness of the raw water, the magnesium content, and the degree of softening to be practiced should be considered in selecting the stabilization process. In general, the use of sulphuric acid for this purpose is justified only in small plants, in which case a solution feeder for applying concentrated acid should be provided. If the degree of stabilization needed is moderate, because of the low caustic alkalinity of the softened water (pH somewhat above 9.4), polyphosphates, such as "Calgon" may be used for this purpose, in which case a solution feeder for applying doses of only 2.0 p.p.m. is adequate.

Carbon dioxide gas ordinarily is used for stabilization by recarbonation. The gas may be secured in the liquefied state in steel cylinders or may be generated by the burning of coke, coal, oil or gas in a special furnace. (three pounds of CO_2 for each pound of coke burned). Provisions must be made for scrubbing the gas for cooling and to remove soot, tar and other impurities. A low pressure compressor of appropriate capacity (0.5 p.p.m. CO_2 required for each 1.0 p.p.m. calcium carbonate changed to bicarbonate), a piping system and headers leading to spray impingement devices, perforated pipes or porous tubes of plates, should be provided to distribute the gas. The devices, pipes, tubes or plates should be submerged at least 8 feet in the effluent end of the sedimentation basin, formed by a perforated stilling wall, or in a flume or conduit providing from 3½ to 10 minutes contact between the water and the gas prior to filtration.

Solid-Contact Softening Basins - As noted in Chapter VII, these basins were developed for the lime-soda softening process, so that coagulation would be favored by the presence of previously precipitated calcium carbonate. The floc so formed is heavy, thus permitting an upward rate of flow in the clarification zone of 1 to 1-1/2 gallons per minute per sq. ft. of surface area at the slurry separation line (top of "slurry blanket"). This in turn permits total detention time in such basins as short as 1 hour, when conditions and operation are favorable. The relatively small size so secured may compensate for the higher costs of such patented basins and equipment, as compared to conventional flocculation and sedimentation basins. The capacity of the plant, the character of the raw water, anticipated chemical doses and degree of softening, and reliability of operation should be weighed before solid-contact basins are selected for any given supply.

The engineering reports should include data supporting the selection of such basins.

The size and shape of these patented basins should be based upon the characteristics of the specific type selected, adapted to local conditions.

BASE-EXCHANGE SOFTENING PLANTS

General - Consideration should be given to the use of this type of softening plant because it is especially suited to the softening of well waters, more likely to be hard than surface waters, and also because it is more easily operated and controlled than a lime-soda softening plant. The raw water, however, should not have a turbidity over 5.0 p.p.m., nor an iron and/or manganese content over 1.5 p.p.m., and this only where not more than 0.5 p.p.m. of the iron and/or manganese is in the oxidized state. If, however, the iron is present as soluble ferrous bicarbonate it can be removed by the base-exchange process even when present in concentrations as high as 10.0 p.p.m., provided air is excluded from the water prior to softening. (See Process 9 - Chapter IX)

Base-exchange softening should not be practiced when the effluent will have a total sodium content in excess of 800.0 to 900.0 p.p.m., otherwise the zeolite will be adversely affected.

Residual chlorine reacts unfavorably with some organic exchange materials, so any required disinfection of the supply with chlorine should be applied to the effluent of such softeners. Inorganic zeolite is damaged when used to soften water containing less than 12.0 p.p.m. silica expressed as silicon dioxide, and also when the carbon dioxide content is very high, nor should silica gel zeolites be used with waters having a pH in excess of 8.4, which fortunately is above the pH of most natural waters.

Base-Exchange Materials - The selection of the base-exchange materials should be based upon the consideration of local conditions including availability, costs, hardness of raw waters, the relative quantity required (size of bed), and the proposed length of operating cycle between regenerations. Base-exchange materials are patented, so detailed information must be secured from the selected manufacturer, so as to determine the size of unit from the grains of hardness removed per cubic foot of material, and also the size of the brine storage from the pounds of salt required per 1,000.0 grains of hardness to be removed. The grains of hardness removed per cubic foot of material per cycle range from 2800.00 to 5500.0 for natural zeolites, from 9000.0 to 12,000.0 for synthetic, inorganic zeolites, and are about 7000.0 for phenolic type, and about 30,000.0 for the non-phenolic type of organic materials. The pounds of salts required per 1000.0 grains of hardness removed varies from 0.25 to 0.35 for organic materials to 0.50 for inorganic zeolites.

Softening Units - Either pressure, or open, gravity units are satisfactory. Futhermore, either upward or downward flow may be utilized, but the latter are more numerous as they permit higher rates of flow without disturbing the bed. The units should not be designed to operate at rates in excess of 6.0 g.p.m. per sq. ft. Back-wash rates of 6 to 8 g.p.m. per sq. ft. are normal. The minimum depth of material in the unit should be 30 inches. The free-board should be about 50% of the depth of bed, but it should be selected with due regard to the specific gravity of the material and the direction of flow. Graded gravel to a depth of at least 15 inches should support the base-exchange material. Each unit should be fitted with a rate-of-flow indicator.

By-pass - A by-pass of the softening units should be provided to permit the blending of raw water with the water softened to zero hardness, so as to secure the desired hardness of the mixed plant effluent. The by-pass should be fitted with a metering device so the proportion of the hard water which is by-passed can be controlled. Consideration should be given to the removal of iron and/or manganese from the by-passed water if the concentration of these minerals in the blended effluent exceeds the desirable limit of 0.30 p.p.m. Reference is made to Chapter IX for a discussion of the factors governing the design of equipment suitable for this purpose.

Salt Storage - Facilities should be provided for the storage of at least 1-1/2 truck-load or one car-load of salt in a basin to which water is added to form a saturated solution. The water level should be float-controlled to admit water when saturated brine is withdrawn. The use of a brine ejector should be considered, so the saturated brine will be diluted as used. The ejector should have a capacity equivalent to 6 to 8 g.p.m. per sq. ft. of bed area. A wash-water connection should be provided for flushing salt from the unit after regeneration.

Corrosion Prevention - If the amount of hard water to be by-passed around the softener units does not produce a blended effluent which is non-corrosive, provisions should be made to add soda ash or caustic soda to the effluent for corrosion control, as these alkali raise the pH as needed, but do not increase the hardness of the treated water. (See Chapter XII)

Waste Brine Disposal - As the calcium and magnesium chloride in the waste brine solution are harmful to vegetation and fish life, the brine should be disposed of in a stream providing a dilution after mixing sufficient to secure a final concentration of not more than 500.0 p.p.m. chlorides in the stream. The waste brine should not be discharged into a pond or lake near wells, otherwise seepage into the waterbearing stratum may seriously increase the hardness and chloride content of the well water.

Controls - Automatic controls for regenerating base-exchange softeners are a convenience, but are not advocated for plants where technical supervision is lacking.

Operation - Facilities should be provided for determining the hardness by the soap method of the raw, softened and blended waters. If corrosion prevention treatment is needed, then facilities also should be provided for determining the alkalinity and pH of samples.

CHAPTER XI. FLUORIDATION

Objectives - The fluoridation of public water supply is designed to provide the optimum concentration of fluorides in the water needed for the development of teeth most resistant to decay among the children served by the supply. The optimum amount is 1.5 mg. per capita per day. Water consumption in temperate climates is about 1.5 liters per capita per day, so a concentration of 1.0 mg. per liter, or 1.0 p.p.m., provides the optimum amount. The consumption of portable water is related to the average, maximum temperatures. The following range in doses is based upon this fact.

Range in Annual Average of Daily Maximum Temperature	Fluoride ion dose - p.p.m.
50 to 54 F.	1.2
54 to 58 F.	1.1
58 to 64 F.	1.0
64 to 70 F.	0.9
70 to 79 F.	0.8
79 to 90 F.	0.7

The dose of 0.8 p.p.m. fluoride ion has been used in Latin America.

The fluoridation of a public water supply, as a dental health measure, usually will be instigated by health officials, with the support of local doctors, dentists and the public. It is then the responsibility of the water supply officials and the designing engineer to establish suitable and effective operation and control of the fluoridation process and to design satisfactory equipment and facilities, which will insure the proper handling of toxic fluoride chemicals and their accurate application to the water supply. This requires teamwork between the doctor, dentist and engineer in establishing a program with proper supervision of suitable equipment.

Fluoride Chemicals - Commercial sodium fluorides, sodium silicofluoride, fluosilicic acid and ammonium silicofluoride have been used in the U. S. for this purpose, but hydrofluoric acid is so active and corrosive that it has been used at only one plant. Fluosilicic acid is a concentrated liquid which must be shipped in glass containers, or rubber lined drums. Ammonium silicofluoride is available only in limited quantities in the USA.

Sodium silicofluoride should be appraised in comparison with sodium fluoride in the light of its advantages: namely, cheaper cost, greater content of fluorine, and its disadvantages; namely its dust producing fine powder form and its much lower solubility. The properties of the two chemicals are indicated in the following tabulation.

DATA AS TO SODIUM FLUORIDE AND SODIUM SILICOFLUORIDE

Name of fluoride compound	- Sodium fluoride	- Sodium silico-fluoride regular
Formula	- NAF	- NA_2SiF_6
Molecular Weight	- 42	- 188.05
Form of fluoride compound	- Fine powder or granular (20-40 mesh)	- Fine powder
Purity, percent fluoride compound	- 95.0	- 99.0
Percent fluoride ion in commercial compound	- 43.0	- 60.0
Pounds fluoride compound per cubic feet	- 75*	- 72**
Pounds fluoride compound per mg/ppm fluoride ion	- 19.3	- 14.0
Percent solubility fluoride compound at 32° F.	- 4.0	- 0.43
Solubility fluoride at 60° F.	- 4.0	- 0.62
Percent pounds per gallon fluoride compound for solution at 60° F.	- 0.43***	- 0.052***
Gallons saturated solution per mg/ppm of fluoride ion	- 58.5***	- 269.2***
Pounds fluoride compound per gallon of 1% solution	- 0.0832	- above saturation value
Gallons 1% solution or slurry per mg/ppm fluoride ion	- 234	- 167
pH of 1% solution	- 6.5	-
pH of saturated solution	- 6.0	- 3.5

* Grades of sodium fluoride carrying weight from 38 to 95 pounds per cu. ft.
** Sodium silicofluoride is available in two grades, namely the regular weighing 72 lbs./cu. ft. and the fluffy weighing 55 lbs./cu. ft.
***Based on saturated solution at 60° F.

Fluosilicic acid may be purchased as a solution frequently containing 23 to 30% of the acid by weight. It is available as a by-product in the manufacture of phosphate rock, Freon gas, etc., and when neutralized becomes sodium silicofluoride. Its properties are as follows:

DATA AS TO FLUOSILICIC ACID OF 30% STRENGTH

Formula H_2SiF_6

Molecular Weight = 144.1

Form = Solution

Purity, percent of compound = 30%*

Percent F ion in 30% solution = 23.7%

Specific gravity solution = 1.27

Weight of 1 gallon = 10.6 pounds

Pounds F ion in 1 gallon = 2.51 pounds

4.22 pounds of 30% solution = 1.00 pounds F ion

pH of 30% solution = 1.2

*Commercial fluosilicic acid solution varies in strength from 23 to 30% of the acid by weight. The above values should be reduced proportionally when acid of less strength than 30% are pruchased.

Fluosilicic acid is shipped in rubber lined steel drums or glass bottles. The acid should be undiluted or it should be diluted with 20 parts or more water to 1 part of acid, otherwise a precipitate will form. Therefore it is best fed directly from the shipping container in a closed system to prevent the release of fumes, and any dilution should be in covered tanks. A hypochlorinator is well suited to the feeding of the acid. Larger volumes may be applied by corrosion-resistant, piston pump type of feeders.

Generally speaking the use of granular sodium fluoride is preferable for small supplies, as solution feeders may be used without the need for dust control equipment, but the much more favorable cost of sodium silicofluoride justifies its use when the required dose exceeds 4 oz. per hour, the reliable minimum capacity of dry chemical feeders. Fluosilicic acid is suitable for supplies of all sizes, and its use is largely a matter of availability and cost.

Chemical Storage - Sodium fluoride and sodium silicofluoride should be stored in unopened shipping containers, unless they are transferred when received to a covered storage hopper or bin. Pneumatic conveyor equipment is advocated for large installations to eliminate the handling of chemicals and the dust hazard. These chemicals are not unduly hygroscopic, but storage facilities should be designed so as to be reasonably dry to prevent the formation of lumps.

The floor surface should be smooth and impervious and should slope to a drain to facilitate wet mopping of any spilled chemicals.

Fluosilicic acid is stored in the shipping containers, or in rubber lined steel tanks.

Scales should be provided and loss-of-weight recorders are desirable at large plants.

Chemical Feeders - Chemical feeders for applying fluoride compounds should be selected with the consideration of the factors discussed in Chapter VI. Proportional feeders should be used when the rate of flow of water being treated varies more than 10%. The maximum capacity of the feeder should not be greater than 150% of the required dose, unless provisions are made to discourage the unauthorized or unnecessary increase in rate of feed of the chemical. Chemical feeders may be altered for this purpose.

The maximum and minimum capacities of chemical feeders vary with the chemical being applied and with the characteristics of each unit. Detailed information is available from the manufacturers.

Generally speaking, presently available equipment provides the choices shown in the following table, although it is not good practice to select a feeder which will have to be operated at its minimum or maximum capacity.

RANGE IN CAPACITY OF THE CHEMICAL FEEDERS

Type of Feeder	Chemical and Strength of Solution	Range in Capacity	Range in Volume of Water treated 1.0 ppm F.
Diaphragm pumps (Hypochlorinators)	Sodium Fluoride 4.0% (saturated)	2.5 to 500.0 ml/min.	43.0 to 8.600 l/min. (Dilute sol. will treat even less)
Diaphragm pumps	Sodium silicofluoride 0.4% (saturated)	2.5 to 500.0 ml/min.	4.3 to 860 l/min (Dilute sol. will treat even less)
Diaphragm pumps	Fluosilicic acid 30% (23.7% F.)	2.5 to 500.0 ml/min.	255.0 to 51000 l/min.
30 liter hypochlorinator with variable head control	Sodium fluoride 2.0%	2.0 to 20.0 ml/min.	34.0 to 340.0 l/min.
30 liter hypochlorinator	Fluosilicic acid 30% (23.7 F.)	2.0 to 20.0 ml/min.	204.0 to 2040.0 l/min.
Volumetric, dry feeders	Sodium Fluoride	0.46 gr. to 36.6 kg/min.	209.0 to 1000.0 l/min.

(Continued)

RANGE IN CAPACITY OF THE CHEMICAL FEEDERS (cont.)

Type of Feeder	Chemical and Strength of Solution	Range in Capacity	Range in Volume of Water treated 1.0 ppm F.
Volumetric, dry feeders	Sodium silicofluoride	0.46 gr. to 36.6 kg/min.	281.5 to 1000.0 l/min.
Gravimetric, dry feeders*	Sodium Fluoride	75.0 gr. to 36.6 kg/min.	33.0 to 1765 m^3/min.
Gravimetric, dry feeders*	Sodium silicofluoride	75.0 gr. to 36.6 kg/min.	46.0 to 2465 m^3/min.

*Gravimetric feeders are preferable for large supplies

NOTE: 1000 ml. = 1 liter = 0.211 Imperial Gallons = 0.264 U. S. Gallons

1 kg. = 2.2 pounds

The volume of water used in preparing a solution of sodium fluoride should be metered or otherwise measured. The pipe supplying dilution water should terminate above the flow line of the tank. The discharge of the solution pump or hypochlorinator, used as a solution feeder, should be fitted with a spring-loaded valve when it is connected to a suction pipe or a conduit which may be under negative pressure, so as to prevent siphonage of the solution into the pipe.

Dry chemical feeders of either the volumetric or gravimetric type are acceptable. They should be completely enclosed, and precautions for dust prevention should be taken, as discussed below. The solution pot should be sufficiently large to provide a detention period of at least 5 minutes when sodium fluoride is to be applied and preferably 15 minutes when sodium silicofluoride is to be applied.

Provisions should be made to supply at least 12 gallons of water per pound of sodium fluoride, or at least 60 gallons of water per pound of sodium silicofluoride being dissolved in the solution pot.

The water should be admitted to the solution pot in a manner so as to prevent back siphonage into the water supply, and provisions should be made to prevent the content of the pot draining or being siphoned into the water supply when the unit is shut down.

Protective Equipment - Each operator should be provided with a pair of rubber gloves and preferably with a dust mask for use while handling dry fluoride chemicals. Facilities should be provided for washing the hands and gloves. Chief emphasis, however, should be placed in facilities for preventing the release of dust.

<u>Dust Control Facilities</u> - Provisions should be made for the disposal of empty bags, drums or barrels either by burning or some other means which will minimize exposure to dust. A metal wheelbarrow or tray should be available for the temporary handling of leaking or damaged bags.

The following procedures should be considered in the light of specific local conditions and needs in the design of dust control facilities:

1. Vacuum pneumatic equipment for drawing powdered material from drums, barrels or cars into elevated covered hoppers, the exhaust air being filtered.

2. A covered hopper, with exhaust fan and air filter having a capacity to provide an air flow of at least 200 feet per minute through the opening in the cover through which the chemical is dumped.

3. An enclosure forming a part of the feeder, into which a bag or small drum of the chemical may be placed before the container is inverted and emptied.

4. A homemade adapter for 125 pounds drums, which will replace the regular cover when the drum is to be emptied, and which will connect tightly to a mating adapter on the top of the hopper, a sliding gate being provided to open the adapter to the hopper after a tight connection has been made.

5. A small capacity dry feeder with a covered hopper, where only a small amount of the chemical is transferred at one time from the shipping container to the hopper through the use of a hand scoop. The use of granular sodium fluoride is preferred with this arrangement.

6. A solution tank containing water into which the chemical is dumped to form the solution.

<u>Controls</u> - The operator should be provided with a special field kit for routine use in determining the concentration of fluorides in the raw and treated waters.

CHAPTER XII. THE CONTROL OF THE CORROSIVENESS OF WATER

Objectives - The costly distribution systems of public water supplies and plumbing systems on private property should be protected from the action of corrosive waters. The use of cement-asbestos pipe, or cement lined, cast iron pipes solves the problem for new systems, but does not afford any protection to metallic plumbing systems. The development of plastic pipe for plumbing system has altered the situation when such pipe became more generally used.

The protection of existing metallic piping, therefore, requires the treatment of corrosive waters. The selection of the treatment procedure should be based upon the character of the water and the degree of corrosiveness, the cost of chemicals and equipment, and the effectiveness of operation. Compromise will be necessary when funds are limited and when only partial protection may be afforded by a procedure feasible under specific local conditions.

Aeration - While aeration will remove carbon dioxide down to a value about 4.5 p.p.m., it also adds dissplved oxygen and hence may increase the corrosiveness of many waters, such as well waters of low carbon dioxide content and initially devoid of oxygen, and surface water from large, deep streams or reservoirs with a large content of organic matter. Therefore, aeration alone should not be considered unless the alkalinity of the water exceeds about 100.0 p.p.m., or unless aeration is accompanied by contact with limestone or calcite, as discussed below. Reference is made to Chapter III for discussion of aerators.

Treatment with Alkali - The relative cost, availability and effectiveness of treatment with lime, soda ash or sodium hydroxide, together with the cost of equipment, should be appraised. Ordinarily, lime should be used for this purpose, because of its availability, lower cost and greater effectiveness and because its calcium content facilitates the formation of calcium carbonate protective film on the pipe surfaces.

Soda ash or caustic soda should not be used for this purpose when alkalinity of the water is less than 35.0 p.p.m. (calcium content too low).

Reference is made to Chapter V for a discussion of chemical feeders. The capacity of the feeder should be at least 50% in excess of that required to feed the dose of the selected alkali needed to raise the pH value and alkalinity of the treated water to the values disclosed as necessary by the "marble test" for clacium carbonate equilibrium of the specific water.

Contact Beds - The exposure of water by contact with crushed limestone or granular calcite repeats on a plant scale the process underlying the "marble test". The limits of practical usefulness of this process are: The zone between those raw waters with an alkalinity of zero together with a carbon dioxide content of 35.0 p.p.m., and raw waters with an alkalinity of 70.0 p.p.m. together with a carbon dioxide content of 7.5 p.p.m., based upon complex physiocochemical factors, which govern the solubility of calcium carbonate in water.

Suspended solid and algae will coat the limestone and thus reduce the rate of solution. Therefore these units are especially suited to clear well water supplies, and where technical supervision is not available.

The rate of solution of limestone or calcite is very slow when equilibrium is approached, so contact beds would have to be uneconomically large to provide sufficient time for full equilibrium to be reached. Therefore, a compromise is to use a minimum period of 1 hour, with finely crushed limestone that will pass a 1/3" screen, as a longer period is required for coarser material.

Inasmuch as the volume of voids in crushed limestone is about 40%, one hour of contact will be available when the rate of flow is about 1.0 g.p.m. per cubic yard of stone. The contact beds may be placed in baffled concrete basins with horizontal flow to eliminate the need for underdrains, or they may be located below spray aerators when the initial carbon dioxide content is over the range given in the limits noted above for waters having alkalinities between zero and 70.0 p.p.m., so that it must be removed in part by aeration.

Provision should be made for adding more stone at intervals as the bed dissolves.

Supplementary Treatment - The effluent of a contact bed may be treated with lime if it is essential to secure the full degree of protection, but in this case the required dose will be reduced proportionally by prior contact with limestone.

Controls of Aeration - Facilities should be provided for making the "marble test" and the determination of the pH and alkalinity of the water before and after aeration, application of alkali or exposure in contact beds.

CHAPTER XIII. LABORATORY FACILITIES

Provisions should be made to provide at all filter plants facilities for laboratory tests needed for control of the treatment processes. The minimum requirements at very small plants, subject to technical supervision by some outside agency, would appear to be provisions for the following tests:

Turbidity
Color
Flocculation characteristics (Jar test)
Residual chlorine

In most cases, however, tests for pH and alkalinity should be included with basic tests noted above, whereby the effectiveness of coagulation, sedimentation, filtration, chlorination and corrosion prevention may be determined.

Tests for iron and manganese are necessary when these minerals are present in the raw water in sufficient concentrations to influence the treatment processes.

Finally the test for fluorides is necessary when flouridation is practiced.

Equipment for physical and chemical testing of water is costly and its proper use requires technical training. Fortunately, however, the control test listed above may be made, to the accuracy needed to control the operation of smaller filtration plants, through the use of kits and chemical reagents, which are ready for use in accordance with special directions.

Reference is made to the "Taylor Water Analyzer" - made by W.A. Taylor and Co., 7300 York Road, Baltimore 4, Maryland; and to the "Hellige Aqui Tester", made by Hellige, Inc., 877 Steward Ave., Garden City, New York, as equipment especially suited for this purpose.

The bacteriological examination of samples of water is made by health authorities and by many in charge of public water supplies. The usual procedures for this purpose are complex and require costly equipment. The Membrane Filter Technique, however, is now available to simplify and expedite bacteriological testing of water, so such equipment should be made available when filtration plant operators are in position to determine thereby the bacteriological quality of raw and treated waters. Portable equipment for this purpose is made by the Millipore Filter Corporation, Bedford, Massachusetts.

Each filtration plant should be provided with the following facilities so that the simple equipment may be properly stored and used.

Small desk and chair
Work bench 6 ft. long, 3 ft. wide and 3 ft. high with a top of wood painted with black, acid-resistant paint, and with enclosed storage cabinets underneath
Bookcase or cabinet for chemicals
Sink with water faucet

(continued)

(cont.)

1 double-outlet, water fixture, to which rubber
  tubing may be connected
1 regular water faucet
1 water faucet with male threads
3 electric outlets
2 gas outlets when gas is available

These facilities may be located at the end of the operating gallery of small plants, but should be in separate room at larger plants.

CHAPTER XIV. A STANDARDIZED DESIGN FOR A SMALL WATER FILTRATION PLANT

Brazil - U. S. Cooperative Health Program
of the
Special Health Service of Brazil
and the
International Cooperation Administration
of the U. S.

Introduction

It is recognized that every effort should be made to develop well water supplies not needing treatment. When, however, surface waters must be used, treatment is necessary. The guides to design, as outlined in this document, provide for the exercise of professional judgement in the selection of details which will secure the most effective and economical treatment of a specific water, the degree and character of treatment being determined by the quality of the raw water and local conditions.

Experience has shown that the basic decisions as to a proposed surface water supply pertain to the following:

a. Anticipated population growth and associated consumption of water.

b. Stream flow characteristics, and minimum rate of flow.

c. Need for an impounding reservoir to insure adequate volume of water for foreseeable future needs during dry periods, and the influence of any proposed storage on the quality of the water.

d. Quality of raw water.

e. Required degree of treatment.

f. Funds available for construction and future maintenance and operation.

g. Capability of treatment plant operators.

The required degree of treatment in general is predicated upon the production of an effluent which is attractive and of safe sanitary quality for drinking purposes. Drinking water standards are discussed in the "Drinking Water Standards" - 1946, U. S. Public Health Service, and also in "International Standards for Drinking Water", published by the World Health Organization.

The limitations of treatment by chlorination alone are discussed in Chapter VIII. The magnitude and the range in the turbidity and/or color of the specific raw water discloses whether a slow sand filter would be effective alone or with preliminary sedimentation without coagulation, but with chlorination. (See Chapters V, IV and VII). Many turbid waters require coagulation and filtration through rapid sand filters, with post-chlorination. Pre-chlorination also is needed with more heavily polluted waters.

Inasmuch as many small rapid sand filtration plants are being built, a decision was made to so design the proposed plant for Pirapora, Minas Gerais, Brazil, that it would serve as an example of an economical and easily maintained and operated plant, which nevertheless would provide all essential functions and treatment processes with equipment made in Brazil. While the shear gates, butterfly valves, etc. may not be manufactured in many developing countries, they are simple and relatively inexpensive when imported.

Inasmuch as the same needs prevail in many other countries where metric units are not used, the following conversion factors are given.

Metric	British	American
1 liter/sec.	0.211 gal/sec.	0.264 gal/sec.
0.63 liter/sec.	0.8 gal/min.	1 gal/min.
1 million liters	211,400 gal.	264,200 gal.
3,780,000 liters	800,000 gal.	1 million gal.
3,780 cu. meters	800,000 gal.	1 million gal.
1 cu. meter	211.4 gal.	264.2 gal.
1 sq. meter	10.76 sq. ft.	10.76 sq. ft.
1 kg.	2.205 pounds	2.205 pounds
0.453 kg.	1 pound	1 pound
453.59 gr.	1 pound	1 pound
1 meter	39.37 inches	39.37 inches

Basis of Design

General - The Pirapora filtration plant requires no complex form-work for concrete. Its design capacity at 2.0 gpm/sq. ft. is 35 l/s, (9.2 gal/sec.) but valves and piping are selected to carry overload of 50% or 52 l/s. The plan is such that the plant may be expanded in the future to 70 l/s. without structural alterations of the original plant, giving an ultimate capacity of 105 l/s. at 50% overload. (70 l/s equals 1104 U.S. gpm or 1,589,760 U. S. gal/24 hrs.)

The basic layout plan may be used for plants having capacities in the range of 10 to 100 l/s merely by increasing the size of the units. Larger plants ordinarily would require special design features and use of more than two filters.

Only two gate valves are used, namely on the wash-water piping, because more economical control of flow under low head may be secured by sluice gates and plug valves. Plug valves are used on the drains for the flocculator and the two sedimentation basins, and the waste wash-water drains from the filters. The rate of flow of raw water and of wash-water is to be regulated by two butterfly valves, which can be set to any degree of opening, as indicated later.

Butterfly valves manufactured in Brazil were not fitted with rubber seats and hence leak to some extent when closed. Therefore, they could not be used in place of gate valves, or alone as float-operated rate controllers. Two float-operated "elbow-action" type valves were used for this purpose, as these may be closed manually when the filters are washed or shut-down. Where butterfly valves with rubber seats are available, they may be used throughout a filter plant in place of gate valves or sluice gates, and alone as float-operated rate controllers to provide economical and easily maintained and operated facilities.

Complete treatment is provided by flocculation, sedimentation, filtration, chlorination, corrosion prevention and fluoridation.

Quality of Raw Water - The São Francisco River at Pirapora varies in quality during the wet season of November to March and the dry season during the remaining months. A sample collected in August 1950 had a low turbidity of 20 units and color of 15 units, an alkalinity of 34 ppm, pH of 7.3 and no iron. The suspended solid content in the wet season is much higher. Experiments disclose that the water coagulates readily. The iron entering the upper tributaries in the oxidized, insoluble state, which settles during the dry season in the river above Pirapora, but during the wet season the total iron content is as high as 4.5 ppm , 4.0 ppm of which is readily removed in the insoluble, oxidized state with turbidity. The manganese content is low, namely 0.16 ppm. This water, therefore, is readily treated by conventional treatment, especially during the dry season.

Laboratory facilities will be limited, but are adequate for control purposes.

Details are given below as to the basis of design of the attached plans. Computations are appended.

1. Pumping Station - The pumping station is adjacent to the filter plant and the filtered water reservoir, and houses the duplicate pumps for both raw and filtered water, thus simplifying piping, electrical equipment, operation and maintenance.

Care was exercised in selecting the raw and filtered water pumps so they will have the desired capacity at the specific heads to be encountered, so that undue throttling of their capacity will not be necessary. Flexibility of operation is provided by the filtered water reservoir having a capacity of 252 cu. m., or 2 hours of filter capacity.

2. <u>Parshall Flume</u> - The raw water force main passes below the chemical feeders and discharges into a Parshall flume, where alum, and lime if needed, will be added. The flume has a six-inch wide throat which gives a capacity between the limits of 1.3 and 110.0 l/s, or sufficient for the future 70 l/s. plant even when operated at a 50% overload. The rate of flow will equal the pump rate, which is subject to adjustment by a manually operated butterfly valve with an arm and quadrant for holding the valve in any desired position. The depth of flow in the flume will be indicated by a float-operated indicator and calibrated scale placed at the critical point with such flumes, from which the discharge may be determined.

Gravity flow to a similar plant could be subject to automatic control by a similar butterfly valve fitted with a weighted arm and connected by cable or rods to a float in the flume.

3. <u>Chemical Storage</u> - An area about 4 x 4 meters, for the storage of chemicals, is provided on the floor with the chemical solution tanks. A hand operated hoist facilitates raising the containers of chemicals to this level.

4. <u>Chemical Feeders</u> - Provisions are made for the use of alum, quick or hydrated lime, calcium or sodium hypochlorite, and sodium silicofluoride.

Alum and lime usually do not remain uniform powders or fine granular materials where high humidities favor the formation of lumps. Furthermore, foreign material frequently is present. Therefore, these two chemicals are more readily applied as a solution of alum and a suspension or slurry of lime. In any case, locally made solution feeders are much cheaper than imported dry-feeders, and thus have been used.

Compressed chlorine gas or liquefied chlorine is conveniently handled in cylinders and may be stored indefinitely. Liquid chlorine, however, is only available from factories in Rio de Janeiro and São Paulo, Brazil, and suitable chlorinators must be imported, and they are expensive and difficult to maintain. This applies to many other countries. Major repairs must be made by a specialist employed by the importer. Therefore, calcium or sodium hypochlorite has been selected for routine use in Brazil, except at large plants where skilled supervision is available and the economy of using liquid chlorine counterbalances the disadvantages. This policy was followed at Pirapora.

Fluoride compounds are not available in many countries, so imported sodium silicofluoride has been selected as most economical to import. Solution feed equipment may be used for sodium silicofluoride, provided the solution tank is large enough to permit the use of a 0.3% solution, its solubility at saturation being only 0.63% at warm water temperatures.

Where fluosilicic acid is available, or may be imported economically it may be more economical to apply this chemical through the use of a chemical pump, as described in Chapter XI. The availability of sodium fluoride would permit the use of solutions up to 4% strength, that is 1.7% F. ion by weight.

The duplicate solution tanks for each of these four chemicals have been given capacities sufficient to permit the solution being mixed only once a day, even when only one tank is in use. The availability of two tanks for each chemical permits prolonger mixing of each chemical without complicating the use of the other tanks during a period of at least 24 hours, or the use of each set of tanks for 12 hours each when the plant is enlarged to 70 l/s.

The solution tanks for alum and lime will be fitted with home-made paddle agitators of wood, which may be turned by the hand wheel at the top of the shaft. A slurry of hydrated lime may be prepared and discharged into a special lime feeder to be described later, or quick lime may be slaked in these tanks and then discharged into the lime feeder. Calcium hypochlorite solution (prepared from special 70% strength material) is so readily made, and sodium hypochlorite solution need only be diluted with water, that a hand stirring paddle is adequate for the special hypchlorinator shown. Sodium silicofluoride (0.3% solution) may be dissolved with the degree of agitation given by the revolving paddles, providedthe agitation is continued for at least 5 minutes.

Solutions of alum, sodium silicofluoride and hypochlorites are distructive to concrete, so the tanks for these chemicals will be coated on the inside with enamel, and plastic hose will be used to conduct the chemicals.

The following table summarizes the data regarding the several pairs of solution tanks.

Chemical	Capacity of each of the dup. tanks in cubic meters	% Solution	Max. dosage kg. for 24 hours	Resulting, max. dosage of desired chemical mg./l at 35 l/sec.
Alum	2.8	5.0	140	52
Calcium hypo-chlorite (70% Cl)	1	1.0% Cl	9.4	3.0 Cl
Sodium silico-fluoride (60% F)	1	0.3 F ion	2.8 F ion	0.9 F ion
Spare	1	----	----	----

Solution of alum and sodium silicofluoride will flow from the solution tanks to low cost, constant-level, orifice boxes on the floor below, which in turn are higher than the Parshall flume or the filtered water reservoir, where the chemicals will be added.

Calcium or sodium hypochlorite solutions are corrosive to the metals used in commercial orifice feeders, which therefore cannot be used as chlorinators. Motor driven hypochlorinators are available in Brazil, but must be imported into many countries and are expensive. Therefore, home-made floating orifices have been adapted to the two tanks for hypochlorite solution, as shown in Figure 5. The effective head is held constant as the solution drains from the tank because the float lowers with solution level. The relative elevation of the orifice and solution level, that is the orifice head, may be changed as desired by increasing or decreasing the weights on the float, thus changing the rate of feed. This can be calibrated by measuring the volume of solution flowing into a measuring cup in some unit of time. Plastic tubing and flexible plastic hose will be used with a hard rubber valve for this unit. Major changes in dose will be secured by changing the strength of solution above or below the usual 1%.

An alternate chlorinator may be selected for this plant, namely the home-made unit based upon a 40 liter bottle and controlled siphonage of undiluted sodium hypochlorite solution(15%), as described in the main text. A 40 liter bottle, however, would be too small for the volume of 1% solution needed.

Lime is so insoluble that the slurry prepared in a mixing tank must be agitated continuously and hence cannot be applied with an orifice box. Therefore, the design provides for the preparation of the slurry of hydrated lime or slaked quicklime in a special commercial slurry feeder of sufficient size to hold each batch of slurry. For instance, one with a hopper 0.33 m^3 in capacity in which 10% slurry is prepared would apply 30 kg. of lime and give a dose of 30 mg/l to 35 l/s for a period of 8 hours. This type of feeder incorporates motor-driven agitators and circulating, adjustable cups, which are filled when passing into the slurry and which discharge their content at two points. One feeder therefore may be used to treat both the raw and filtered water with independently controlled doses of lime, and hence may be used to aid coagulation and/or prevent corrosion.

The properties of water treatment chemicals are summarized in the main text.

5. <u>Flocculators</u> - Although experience has shown that motor-driven paddles provide the most flexible and effective agitation, such equipment is expensive and presents a maintenance problem. Generally used baffled basins impose an appreciable loss-of-head when sufficiently large to provide long flocculation periods, and the degree of agitation cannot be altered without structural changes. Therefore, the Pirapora design is based upon using the energy of the flowing water to create alternately upward and downward, helical flow in a series of 5 basins having a total capacity of 60 m^3 to give a detention period of 28.6 minutes at 35 l/s. Agitation is caused by the square plan of each basin, which causes resistance to helical flow.

Rapid mixing of the chemically treated water is provided in the Parshall flume. Progressively lessening agitation is provided in the five basins by the use of locally-made sluice gates to control the size of the openings through which water enters each basin to cause jet action. The 14" pipe from the flume to the first basin was selected to give a velocity of 0.3 m/s at 35 l/s, but this is to be increased to 0.5 m/s by the use of a removable metal plate with orifice. This provides the choice of velocity between 0.3 and 0.5 m/s. The square openings between the other basins in turn may be reduced in size by moving the sluice gates to increase the velocity from the minimum of 0.2 m/s to any selected higher values, but it is anticipated that the lower velocities will be most effective in the last three basins.

The loss-of-head in the flocculator cannot be computed accurately, but is estimated to be only 0.1 m. for the velocities mentioned, by the formula

$$h_f = \frac{Kv^2}{2g} \quad \text{with } K = 1.$$

A small value was added to allow for turbulence.

A drain is provided for one basin together with small sluice gates between the basins to permit all being drained for cleaning. This drain and the sluice gates may be eliminated, if a small, portable pump is available to dewater the basins for repairs or cleaning.

The flocculating facilities, therefore, permit prolonged and controlled agitation without mechanical equipment and without any serious loss-of-head. In fact the pumping loss due to this added head is less than the power requirements of mechincal flocculators.

Five additional basins will be constructed to operate in parallel with the first group, when the plant is enlarged to 70 l/s in the future.

6. Sedimentation Basins - Two sedimentation basins, each 17 m long by 5 m wide and 3.15 m deep, are provided to give a settling period of 4 hours, at 35 l/s, with the lower 0.15 m being reserved for sludge storage. Each basin has a shallow depth of only 3 meters and a length over 3 times its width. The "overflow rate", based on area, is about 17 $m^3/d/m^2$ or 400 gpd/sq. ft. The average velocity of flow is 7 cm/min. or 0.23 ft/min. All of these values are conservative.

Flocculated water will flow to the two sedimentation basins through an open conduit designed to carry 70 l/s at a maximum velocity of 0.4 m/s. Simple alternations will permit the flocculated water from the five future basins to enter this conduit. The water is brought to a central point at the entrance to the two basins, to equalize head loss and hence the rate of flow into each basin. Minor adjustments may be made with the sluice gates. This conduit may be extended to the two future basins without structural change.

Water flowing through each sluice gate enters the separate influent trough of each basin. Each trough is perforated in the bottom with a series of five openings of selected increase in size to secure uniform, downward discharge of the water throughout the width of each basin, in spite of the progressive lowering of the head of the water in each trough. These openings vary in size as follows: 160 cm^2, 180 cm^2, 200 cm^2, 220 cm^2, and 240 cm^2 to secure an estimated uniform flow rate of 0.17 m/s., when both basins are in use.

The downward direction of flow through these openings favors the dissipation of the entering velocity without strong horizontal currents. The stilling wall is 2.5 m from the entrance end of each basin, so eddy currents are dissipated in the large volume of water provided, namely 37.5 m3 or about 36 minutes detention at 17.5 l/s. Any delayed flocculation will be facilitated by the gentle agitation in this zone.

In the past the perforations in stilling walls were located in the upper 75% of the walls or throughout their full depth. Experience with upward-flow, solid-contact basins, however, has shown the value of introducing the flocculated water in the lower portion of sedimentation basins. This is accomplished with horizontal basins by the design under discussion, where the perforations in the wall direct the water into the lower two-thirds of the basins. This also prevents warm water from short-circuiting the basin over the surface when the underlying water is cooler.

Three rows of five openings each are located 65 cm., 115 cm. and 165 cm. above the bottom, so the lowest row will be at least 50 cm., above the sludge. The area of the openings were selected to impose sufficient resistance to flow as to result in its uniform distribution throughout the area of the wall containing perforations, without the rate of flow through the openings being unduly high. Practice is to use velocities from about 0.12 m to 0.24 m per second (0.4 to 0.8 ft/s.). The selected total area shown on the plans is 15 times 100 cm^2 or 0.15 m^2, giving a maximum velocity through the openings of 0.35 m/s., when one basin is operated at the 50% overload at 52.5 l/s. Ordinarily the velocity will be 0.117 m/s., with both basins operated at the total of 35 l/s.

The water flowing through the openings will gradually rise a vertical distance of from 1.5 to 2.5 m while it flows 13.5 m to the outlet wiers. This vertical component of the flow (2.5 m in 4 hours) naturally is slower than the rate of sedimentation.

In fact the sedimentation of the floc through the slowing rising water creates the effect of "solid contact basins", where the settling floc particles increase in size by contact and agglomeration with the lighter floc which otherwise would stay in suspension and slowly rise with the water.

Outlet weirs are in the form of a flume placed 1 m from the end of each basin, giving a double weir with a total length of 20 m. The loading is only 1.75 l/s/m, as contrasted to 7.3 l/s/m generally used as the upper limit (50,000 gpd/ft.), to insure a moderate velocity of approach to the weir. Split, cement-asbestos pipe would serve as low cost weirs for smaller basins.

The bottom of each basin is flat, because a slope is not needed with such small basins with central drains, and the flat bottom simplifies construction. A plug valve is used to control the emptying of each basin. The drain is 10 inches in diameter, which will permit one basin being drained in about one hour, so it will be out of operation for less than 24 hours.

The drain will extend to the edge of the basin, so it may be extended to serve the additional two basins when they are constructed in the future, without altering the concrete work.

The concrete bottom is designed in part to serve as beams to reinforce the side walls. Beams also are used for the same purpose at the upper edge of the walls, and cross-beams provide additional support. Ladders are available for use when removing sludge.

7. Filters - Two 3 x 4.3 m filters are used with a total area of 25.8 m^2, thus providing a normal rate of 35 l/s or about 115 $m^3/m^2/day$ (2 gpm/sq. ft.). As stated, piping sizes have been selected to provide for an overload of 50% or 52 l/s. without undue loss-of-head. Deep filter structures (4 m) are used to provide a water depth of about 2.25 m to increase length of filter runs without undue negative head with its resulting difficulty from air-binding. A shaded light is to be so located that the character of the settled water on the filter may be observed at night.

The unique aspect of the filters is the use of float-operated valves as economical rates controllers, as described later. This requires provisions for equalizing the rate of flow of settled water into each filter within the range of its float. This is provided by having the settled water enter each filter over a circular weir created by a 90° pipe bend. The elevation of the weir is above the maximum elevation of water on the filter. Flow is shut off by an inexpensive sluice gate for each unit.

The two wash-water troughs are ample in size to permit washing at the high rate 1.4 m/min. rise, or a total of 17.64 m^3/min., for either filter. This is in excess of the anticipated maximum rate of 1.2 m/min. (48 inches/min.) to give 50% sand expansion even with warm water. The top of the troughs is 1.35 m above the sand. The bottoms of the troughs are above the level of the expanded sand even with 50% expansion. (Split, cement-asbestos pipe would serve as excellent troughs for smaller filters.)

The sand bed is 0.75 m deep to insure effective filtration through somewhat coarse sand, with an effective size of 0.5 mm., needed to permit operation at 50% overload without unduly short filter runs. Finer sand would collect most of the floc on its surface and form a dense layer, rather than permit a certain degree of penetration found desirable for a more gradual increase in loss-of-head and longer filter runs.

The gravel layer, supporting the sand, is in conformity with general practice.

The filters have false bottoms, in which are located procelain strainers on 20 cm. centers, found by long experience to provide simple and non-corrodable facilities for equal distribution of wash water. No main headers are needed, and the filtered water pipe and wash-water pipe are kept separate by this design.

The bottom of the wash-water tank is 10 m above the wash-water troughs, creating sufficient head to produce the wash-water flows noted above, through a pipe 12 inches in diameter, and of short length, as the tank is located adjacent to the end of the pipe gallery. Other elevations would be used with different hydraulic conditions. This head is sufficient to require the use of a gate valve on each wash-water line. These gate valves will be completely opened or closed when in use, as the rate of flow of wash water is to be adjusted by a butterfly valve with a quadrant, shown on the wash-water piping, thus permitting adjustment to the rate of flow as determined by trial. In this way the rate of flow does not have to be adjusted every time the valves are operated. The rate of flow may be measured by timing the rate of rise of wash water with a double, hook-gauge. For instance a rise of 30 cm. per 15 sec. equals the rate of rise of 1.2 m/min. (Tight-shutting, butterfly valves may be substituted for the two gate valves, when available.)

The wash-water troughs discharge into a "gullet" which in turn is drained through a plug valve to the main drain in the pipe gallery. This arrangement has the following advantages: (a) low priced valve is used; (b) the valve is located at the bottom of the gullet; (c) the layout of piping in the pipe gallery is simplified; (d) the operating floor is freed of two valve pedestals.

The filtered water will flow from under the false bottom of each filter through a separate, 8 inch diameter pipe to avoid the expansive tee otherwise needed to connect the filtered water pipe to the wash-water piping in the usual manner.

It is recognized that conventional rate controllers may be adjusted to provide the desired, uniform rate of filtration through each filter, irrespective of the level of water over the sand or the loss-of-head through the sand. They are expensive, however, and are difficult to repair. Therefore, the design under discussion utilizes for each filter a float-operated butterfly valve fitted with an arm, which is connected by metal rod and pivoted arm or cable over pulleys to a float on the water surface of the filter. The rate of filtration thereby is maintained equal to the rate of flow of settled water into each filter, and this in turn equals the rate of flow of water through the Parshall flume. As stated previously equal flow through each filter must be assured by carefully installing each pipe bend used as a weir to have the same elevation. This elevation must be slightly higher than the maximum elevation of the water surface, so as to insure free weir action. This is accomplished by adjusting the length of the rod or cable from the float to the arm of the valve, so that the valve is in the open position when the loss-of-head is greatest, and the float is so located as to control the elevation of the water at a level just below that of the weir.

Conversely, when the loss-of-head is at a minimum, just after a filter is washed, the rate of filtration will tend to increase, thus lowering the water level and float, which in turn will partly close the valve until equilibrium is established.

When one filter is shut down for repairs, all of the settled water will flow to the other filter, thus doubling the unit rate of filtration. This can be prevented by adjusting the rate of flow of raw water to 17.5 l/s by partly closing the butterfly valve near Parshall flume.

The same hydraulic conditions prevail when one of the filters is washed, but the short period of washing justifies the practice of allowing normal flow and treatment of the raw water, the surplus being stored in the sedimentation basin while the influent sluice gate of the filter being washed is closed temporarily.

The sluice gate of the filter in use of course must be partly closed during this period to reduce the flow of settled water to about 17.5 l/s., otherwise this filter would operate at the rate of 35 l/s.

Inasmuch as experience has shown that most rate controllers are not properly maintained at small plants, and as it is difficult to teach operators how to maintain and repair rate-of-flow indicators and recorders, the availability of butterfly valves thus provide an economical alternate.

The float operated, butterfly valves, to be used as rate controllers, now available in Brazil, do not close tightly and thus must be used with gate valves. Tightly closing butterfly valves may be used alone.

The loss-of-head through the filter is to be measured by the difference in water level in two glass tubes connected by rubber tubing respectively to the filter structure above the sand level and to the effluent pipe between the filter and the butterfly valve.

8. Pipe Gallery - The layout of the piping is so compact that a gallery only 3 meters in width would have been adequate. A 4 meter width was used to provide more room in the overlying structure. A clear well cannot be located beneath the pipe gallery at Pirapora, because bedrock was close to the surface. Therefore, a separate, shallow reservoir is shown. Local conditions would control the choice of location at other plants.

The waste wash water discharges through plug valves and a short length of 16 inch diameter piping into a drain under the floor of the pipe gallery. This in turn connects with the drain from the basins at a manhole, from which a main drain leads to the river. Quick opening plug valves would have been used here, but they are not available in size required, so the regular plug valves were used.

9. Filtered Water Reservoir - This reservoir is to have a capacity of 242 m^3, or 2 hours storage at the filter rate of 35 l/s. The existing elevated tank at Pirapora has a capacity of 212 m^3,

and another storage tank has a capacity of 350 m^3, so the local storage is adequate to provide flexibility in operation.

The new reservoir is adjacent to the filter plant and pumping station to conserve piping and facilitate operation.

A shaded light is to be located in this reservoir to permit the filtered water being viewed against a black and white disc placed on the bottom below the light, as this discloses very low turbidity or residual floc in the filtered water more readily than even laboratory tests.

The wooden baffle shown on the plans of the reservoirs is to prevent short circuiting of the water from the filter discharge pipe to the suction pipes of the high-lift pumps, so that the full capacity of the reservoir is available for the reaction period for chlorine, or up to 2 hours. The addition of lime for corrosion prevention is delayed to a point near the suction pipes, to permit the chlorine to be most active as a disinfectant in the water of low pH value for most of the two hours before lime treatment raises the pH.

10. Laboratory Facilities - Provisions for essential chemical tests are located at one end of the control gallery. The use of a "Taylor Water Analyser" facilitates the tests for color, pH, iron, manganese and fluorides. Turbidity standards in 500 ml. bottles will be provided. Equipment for titrating alkalinity and hardness also will be provided. A homemade flocculator will be furnished to permit the selection of optimum coagulant doses by the jar test.

Storage space is sufficient for equipment for bacteriological examination of samples by the membrane filter techniques.

A toilet is located on the operating floor, and a shower is located on the level of the pipe gallery.

11. Computations -

a. Force main and butterfly valve. Specified to carry 70 l/s.

b. Parshall flume. Size with throat 3 inches wide which is adequate for measuring flows 1.3 l/s up to 110 l/s. All dimensions were taken from Technical Bulletin 136 - L1A published by Builders Providence, Inc., Providence, R.I.

c. Flocculator. Main factors were detention time and agitation velocity. The flocculator is composed of five chambers with inside dimensions of 2.0 by 2.0 by 3.25 m. each. This corresponds to five chambers of 13 cu.m. each, or a total of 65 cu.m., providing a total detention of 30.9 min. The velocities can be varied from 0.13 m/s up to the value necessary for good flocculation since there is a wooden sluice gate for each basin that can be regulated. The loss-of-head of about 0.1 m. was calculated as that through an orifice, that is $hf = k\, v^2/2g$, with $k = 3.3$. The value of v is variable and depends on the position of the sluice gate.

d. Influent Channel. Designed for 70 l/s in such a way that the flocs neither will be broken nor settled. The cross sectional area is $(0.50 \times 0.50) m^2$ which gives a mean velocity flow of 0.14 m/s for a present plant capacity of 35 l/s, or 0.28 m/s for double this volume.

e. Submerged Orifices. They have selected, increased areas in order to discharge approximately the same amount of water under the decreasing heads in the trough. The total orifices area is $0.192\ m^2$.

f. Stilling Wall. The determination of the total orifice area was intended to avoid breaking the flocs when only one settling tank is working and the plant is 50% overloaded, when the maximum velocity is 0.25 m/s. Each stilling wall has 3 lines of 5 orifices. The orifice height is 0.15 m and the width is varied; the top row is 0.08 m wide, the middle one is 0.09 m and the bottom row is 0.10 m.

g. Weir at the end of the sedimentation tank. Designed for 2 l/s per m of length.

h. Filter Inlet. Pipe chosen in such a way that the flocs will not be broken at the design velocity of 0.20 m/s, with one filter working at 50% overload.

i. Other filter computations. Filter area - Based on 2 gal/sq. ft. per min. as rate of filtration. Filter bottom composed of porcelain strainers with 4 holes of 1.5 cm x 1 cm each. The number of strainers is 25 per sq. m.

j. Backwashing water rate. This rate was determined by the formula:

$$v = 0.762 \times d^{3/2} (1 + 0.06\ E)\ (0.03\ t + 0.7)$$ the units being:

v = rise in m/min; t = temp. ^{o}C; d = effective size of sand in mm.

E = sand expansion = % of expansion is to normal depth of sand.

Then, with 50% expansion, a temperature of 21°C. and effective size of 0.50 min., the velocity is computed to be 1.43 m/min.

Then, from the size of each filter, the discharge = $4.5\ m \times 6.5\ m \times 1.43\ m/min = 41.8\ m^3/min = 696.6\ l/s$.

k. Loss-of-head during backwashing, with wash-water tank at minimum depth of water.

1. Through sand and measured to the upper level of the sand bed.

$$h/L = (s_s - 1)\ (1 - f_e);\ \text{and}\ f_e = (v/v_s)^{0.22}$$

Units and meaning of the symbols:

v_s = settling velocity, cm/s

v = backwashing velocity (cm/s) = 1.43 m/min = 2.383 cm/s

f_e = porosity ratio

s_s = sand specific gravity = 2.65

h = loss of head at sand layer, m

L = sand depth, m

Since the sieve analysis is unknown, v_s cannot be determined for each sand size. However, as an average v_s = 30 cm/s may be used.

$f_e = (2.383/30)^{0.22} = 0.5728$

$h/L = (2.65 - 1)\ (1 - 0.573) = 0.7046$

$h = 0.7046 \times 0.75$ m = 0.53 m

2. Loss-of-head through gravel

h = loss-of-head; D = depth of gravel

$h = 0.12 \times D$; so $h = 0.12 \times 0.75 = 0.09$ m

3. Loss-of-head through strainers

Useful strainer orifice area assumed to be 75% so that

$0.75 \times 1.5 \times 1 \times 4 = 4.5\ cm^2$ per strainer.

No. of strainers per m^2 = 25.

Total useful area = $25 \times 4.5 = 112.5\ cm^2$.

Back wash discharge per sq. m = 2.383 cm/s x 10,000 cm^2 =

23,830 cm^3/s.

Jet velocity v = 23,830 $(cm^3/s)/112.5\ cm^2$ = 211.82 cm/s =

2.12 m/s.

Loss-of-head $h = k\ v^2/2g = 3.3 \times 2.12^2/19.6 = 0.79$ m.

4. <u>At drain inlet</u>

$h = f\ v^2/2g = 0.23$ m

5. <u>At pipe outlet</u>

$h = 0.46$ m

6. <u>At piping and specials</u>

It depends on piping length and specials used.

The wash-water tank bottom should be above the lip of the filter gutter a height equal to the sum of all these items 1 through 6, or 2.10 m plus the estimated loss in the piping and specials.

l. <u>Backwash water tank.</u>

Maximum rate - 1.4 m/min   filter area - (3×4.2) m^2

Time of backwashing - 5 min.

Time allowed for valves operation - 2 min.

Volume - 1.4 m/min x (3×4.2) m^2 x 7 min = 123.48 m^3

Volume for plant operation - 6.52 m^3

Total volume - 130 m^3

m. <u>Solution tanks.</u> Enough for 24-hour continuous operation

Two - each 1.95 x 1.30 x 1.10 meters = 2.8 m^3

Hypochlorite, fluoride and spare - Two each, or six in all -

1.40 x 0.60 x 1.10 meters = 0.92 m^3

The freeboard is 0.1 m, in tanks having total depth of 1.2 m.

The maximum dosages were assumed to be:

Alum - 30 ppm

Lime - 30 ppm

Hypochlorite - 3 ppm

Sodium silicofluoride 1.13 ppm = 0.8 ppm F ion.